CORPS OF ENGINEERS, U. S. ARMY

BANK CAVING INVESTIGATIONS
MORVILLE REVETMENT, MISSISSIPPI RIVER

TECHNICAL MEMORANDUM NO. 3-318

CONDUCTED FOR

THE PRESIDENT, MISSISSIPPI RIVER COMMISSION
CORPS OF ENGINEERS, U. S. ARMY

BY

WATERWAYS EXPERIMENT STATION
VICKSBURG, MISSISSIPPI

SEPTEMBER 1950

Report Documentation Page

Form Approved
OMB No. 0704-0188

1. REPORT DATE **SEP 1950**	2. REPORT TYPE	3. DATES COVERED **00-00-1950 to 00-00-1950**
4. TITLE AND SUBTITLE **Bank Caving Investigations, Morville Revetment, Mississippi River**		5a. CONTRACT NUMBER 5b. GRANT NUMBER 5c. PROGRAM ELEMENT NUMBER
6. AUTHOR(S)		5d. PROJECT NUMBER 5e. TASK NUMBER 5f. WORK UNIT NUMBER
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) **U.S. Army Corps of Engineers,Waterway Experiment Station,3903 Halls Ferry Road,Vicksburg,MS,39180**		8. PERFORMING ORGANIZATION REPORT NUMBER
9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)		10. SPONSOR/MONITOR'S ACRONYM(S) 11. SPONSOR/MONITOR'S REPORT NUMBER(S)
12. DISTRIBUTION/AVAILABILITY STATEMENT **Approved for public release; distribution unlimited**		
13. SUPPLEMENTARY NOTES		
14. ABSTRACT		
15. SUBJECT TERMS		

16. SECURITY CLASSIFICATION OF:			17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES	19a. NAME OF RESPONSIBLE PERSON
a REPORT **unclassified**	b ABSTRACT **unclassified**	c THIS PAGE **unclassified**	**Same as Report (SAR)**	**117**	

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CONTENTS

CONTENTS (cont'd)

BANK CAVING INVESTIGATIONS

MORVILLE REVETMENT, MISSISSIPPI RIVER

PART I: INTRODUCTION

Purpose and Scope of Investigation

1. This report presents the results of a soils investigation made at Morville Revetment, Mississippi River, to determine the cause of a large failure which occurred in the revetted bank on or about 30 May 1949, and of subsequent minor failures and bank slumping. The revetment is located on the right bank of the river about 7 miles below Vidalia, Louisiana, at approximately river mile 352 AHP (Index Map, plate 23). Preliminary studies of the failure area indicated that certain phenomena pertaining to the original failure, such as size, shape, and the time element involved, were characteristic of other large failures which have occurred along the Mississippi River. Liquefaction is thought to be a major factor in these failures and the determination of the in-place densities of the sands was an important part of this investigation.

2. Seventeen soil borings were made in this investigation to determine the character of the soils in the project area and to define geological formations. Undisturbed samples were obtained for laboratory testing. Three borings were made from floating plant in an effort to establish the presence of revetment in the failure scar. The results of field and laboratory tests are given in the main body of this report and details of procedures are included as an appendix.

Bank Recession at Morville

Bank recession prior to 1949

3. After the opening of Giles Cutoff (mile 363 AHP) in May 1933, a gradual recession occurred in the right bank of the river at Natchez Island Towhead (mile 350 to 357 AHP) which averaged approximately 120 ft a year for the 10-year period between 1934 and 1944. The action increased after 1944 and the right bank receded a maximum distance of about 1900 ft in the two-year period 1944 to 1946, and about 1000 ft in the three-year period 1946 to 1949. Plate 1 shows the movement of the mean low water contours on the right and left banks from 1934 to 1949. The soils of Natchez Island Towhead are of an erodible type of river deposit in that they are predominantly silts and sands and apparently do not contain any clays which might be more resistant to bank erosion. The silty and sandy nature of the upper-bank soils on Natchez Island is shown by plates 2 and 3.

4. Due to the recession of Natchez Island, it became apparent that a revetment was needed at Morville to protect the exposed point of the levee in that area, as shown on plate 1. Consequently, Morville Revetment was constructed in December 1948 and January 1949. The underwater slope, below approximately 48 ft msl, was protected with dumped mass asphalt. A large bank failure occurred in the area between ranges 12-D and 25-D (plate 5) during the grading operations. An exposed point was left at about range 12-D which was not removed by bank grading; this point and the resulting cove are shown by the contours of the after-construction survey of January 1949 (plate 5). The upper bank was

neither paved nor graded.

Hydrographic surveys

5. Hydrographic surveys were made in the area, by the New Orleans District, CE, following revetment construction. The surveys consisted of soundings made at regular intervals along each range. The results of the surveys are shown as contour maps, constructed from the range surveys, on the plates discussed in the following paragraphs. The bank changes that occurred between surveys are shown as cut-and-fill maps. The cut and fill that occurred on any given range was computed from the basic data rather than from the contour maps, since the contour maps are themselves constructed from the range surveys.

January to 16 May 1949 before-failure survey

6. A stage hydrograph for the Natchez Bridge gage for the first half of 1949 is shown as plate 4. Periodic hydrographic surveys were made in the area during the high-water season, January to 16 May, following construction of the revetment and are presented on plates 5, 6, 7, and 8. No serious cut or fill occurred in the revetted area during this time, although deepening in the order of 25 ft in the thalweg slightly upstream of the upper end of the revetment and at the extreme upper and lower ends of the revetment did occur, as shown on the cut-and-fill map on plate 8. In the locations where failures later occurred the deepening was not more than 5 ft and occurred in only a few local areas. Surveys were made almost weekly during April and the first part of May 1949, because it was thought that the stability of the bank would

be lowest during the rapid fall of the river (plate 4). The river ceased to fall between May 20-25 and then began to rise again. The revetment was considered to be temporarily out of danger and the weekly hydrographic surveys were stopped.

30 May 1949 failure,
and after-failure survey

7. Some time between May 28 and 30, a failure occurred at the exposed point on range 10-D and involved a soil mass of approximately 350,000 cu yd. The sketch on plate 9 shows the location of this failure and also the subsequent failures discussed on the following pages. The survey of 31 May (plate 10), immediately following the failure, showed only a small amount of fill in the area offshore and downstream of the failure, indicating that almost the entire mass had moved out in a downstream direction and had been carried away by the river. The maximum deepening that had occurred in the channel in the failure area consisted of an extension of the previous deepening at the upper end of the revetment. While there may have been some deepening at the toe of the failure which was later masked by the fill from the failure, such deepening, if it did occur, was probably small, since the deepening for some distance upstream was only about 5 ft or less.

8. The failure was located at the upstream edge of the earlier failure which had occurred during construction, and may properly be considered an extension thereof. It carried away the exposed point left by the first failure, but another point remained on range 7-D and the cove was enlarged. Plate 11 is a picture of the cove, taken several days after the failure from the exposed point on range 7-D. No marked

turbulence was visible in the area shortly after the failure had occurred, and the water immediately surrounding the area was extremely quiet as evidenced by plate 11.

5 June 1949 survey

9. The survey of 5 June 1949 (plate 12) did not reveal any major changes in the area other than removal of the fill that had been found after the failure of 30 May, indicating that the river had not attacked the bank area during that time.

7-11 June 1949, continuation of failure

10. On 6 June 1949, the exposed point at range 7-D failed and the entire upstream bank of the failure area was carried away back to Range 5-D, as shown on plate 13. The failed material apparently moved out almost perpendicularly to the flow of the river in this area, as evidenced by the fill shown on plate 12, rather than in a downstream direction as the previous failure had done. No other changes were evident in the area. An exposed point was left on range 5-D and the cove was considerably enlarged by this failure. The formation of an exposed point after each failure and the resulting pocket which developed behind these points indicate that the riverward portion of the bank which formed the point was somewhat resistant to failure. The nature of this relatively hard point was determined in the course of the investigation and is discussed later.

Repair measures

11. Decision was made that the revetment should be held and that

repairs should be initiated at once. The repairs consisted of grading the upper bank and placing a single thickness of articulated concrete mattress over the entire area and double thickness in the area where bank failures had occurred. The mattress in the failure area extended from the water surface to the thalweg of the river, a distance of more than 400 ft. Complete surveys could not be obtained after 11 June because of the combined bank grading and paving operation. Bank grading was commenced on about 9 June at the downstream end of the revetment and progressed upstream. The bank was never successfully graded at ranges 5-D to 8-D because the material in this portion of the failure area adjacent to the exposed point slumped in the river as fast as it was cut, and a graded slope never existed for more than a few hours. The exposed point at range 5-D was partially degraded and was reported by the foreman of the bank grading unit to show considerable resistance to the grading operations. Nature of operations? ?

11-13 June 1949 failures

Groundwater should be shown on all cross-sections.

12. No surveys were made during this time. Bank seepage was occurring in the area and was causing some minor upper bank slumping (plate 14). The river became extremely turbulent, possibly due in part to the repair activities of barges and towboats, and a strong upstream eddy formed in the cove. On 12 June a small failure occurred at ranges 13-D to 14-D. The failure exposed the soil strata for some distance back into the bank. All of the strata appeared to slope away from the river. The slope of the strata and the nature of the soils exposed are shown by plates 15 and 16. On 13 June a small failure, involving

most likely cause.

approximately 50,000 cu yd, occurred between ranges 18-D and 21-D. The soil strata exposed by this failure also appeared to slope away from the river. The general appearance of the failure is shown by plate 17.

<u>14-24 June survey</u>

13. The survey of 14-24 June 1949 (plate 18) indicates the limited extent of the underwater portion of the failure at ranges 13-D and 14-D, and the greater deepening caused by the failure occurring between ranges 18-D and 21-D. There was almost continuous bank slumping in the upstream portion of the cove. Extensive, but generally minor, bank slumping occurred in the vicinity of range 3-D as shown by plates 19-22. There was no serious deepening of the thalweg except at the upper end of the revetment where increases in depth ranged up to 25 ft. There was some fill on range 12-D which probably was caused by the bank slumping near the exposed point. A complete survey of the upstream portion of the cove was not possible because of bank-paving operations. On 18 June a failure occurred between ranges 5-D and 7-D which carried away most of the remaining point and a considerable portion of the upstream bank of the cove. The top of bank after failure is shown on plate 18, survey of 14 to 24 June. A resistant ridge remained under water extending downstream from the waters edge at range 0 to range 5-D. Some parts of the ridge were visible at lower river stages on 18 August and asphalt could be seen, although it could not be determined whether this asphalt was part of an intact mat or merely a fragment. This ridge was under continuous attack by the river and the greater part had been removed by 15 September 1949.

PART II: FIELD AND LABORATORY INVESTIGATIONS

Field Investigation

Plan of borings

14. The plan of borings is shown on plate 23. Borings S1, S2 and S3 were made from a barge, anchored in the cove, to determine if the original mat still remained in the area. Boring R10-1 was an exploratory boring made to determine the general soil conditions. This boring revealed that the overburden consisted predominantly of silty soils and was about 40 ft in thickness. This information, combined with the manner in which the bank caving had been occurring and with data from previous geological studies, indicated that the area was a point-bar deposit, probably made up of alternating ridges and swales, which ran approximately parallel to the river and Natchez Island Chute. It appeared that the main portion of each failure was occurring in a sand ridge and the relatively hard exposed point which was left after each failure was due to a swale filling which acted as a deeper, more cohesive overburden on the underlying sand. Proof. It was decided to locate borings in two locations upstream of the failure proper. These borings were intended to furnish soil profiles across the ridge and swales to a depth of 150 ft or to the top of the Tertiary formation whichever provided for the shallower boring. Borings R2-1 to -4 and R1U-1 to -4 were planned accordingly.

15. Backswamp deposits, which might be more resistant to the action of the river, had been mapped in the general area. Borings R2U-1 and R2U-2 were made to determine the approximate boundary

between the point bar and backswamp soils. The borings were to be advanced to a depth of 150 ft or to the Tertiary formation.

16. The remainder of the borings were located along the river bank to determine the variation of the soils parallel to the river and to investigate the possibility that loose sands might underlie the entire area rather than exist only in the ridge described above.

Method of sampling

17. It was considered essential that the borings indicate relative values of density of the sand in the various areas under investigation, in order to provide a basis for determining the likelihood of further bank caving. It was decided to obtain this information by driving a 3-in. piston sampler into the soil and measuring the penetration resistance of the soil to the sampler by means of a gage on the hydraulic system of the drilling rigs. It was thought that undisturbed samples of the clean sands probably could not be recovered and that the penetration resistance measurements and data from bailer or calyx samples would be the maximum information that could be realized. Frozen plug sampling of the clean sands was known, from previous experience, to be too slow for use in the limited time available. It was on this basis, therefore, that the boring program discussed above was initiated. Early in the program, however, the boring crews developed a procedure whereby undisturbed samples of clean sands could be obtained in a reasonable length of time. This procedure entirely eliminated casing and used commercial drilling mud instead. A description of this new method of field exploration and details of the samples obtained thereby are presented in

an appendix to this report. One hundred and ninety-eight (198) sand samples were obtained. Only three samples were lost using this method, and these losses occurred in the first boring made (R2-2) because the drilling mud was not thick enough. The proportion of drilling mud to water was increased thereafter.

18. Samples obtained by the above method were 3 in. in diameter and 2 to 2.5 ft in length and were contained in a seamless steel (Shelby) tube. The following procedure was followed in examining samples in the field. The sand was removed from the sampling tube in increments by means of a small cup-auger previously developed for this purpose in connection with frozen plug borings, and the density of each increment was determined. This procedure was carried out in the field to eliminate any possible disturbance due to handling and transportation. Samples to be examined in the laboratory were sealed at both ends and carefully packed in sawdust and wood shavings and shipped to the laboratory.

Observations during sampling

19. Considerable importance was attached to the penetration resistance of the sampler as a means of estimating the nature of the soil in cases where a sample was not recovered, and for this reason the time in seconds and the force required to make each drive were recorded for every sample taken, including those in the overburden materials. The time required for each drive was determined with a stop watch. The total force necessary to make each drive was measured by a Bourdon gage connected to the hydraulic system of the drill rig, and was read for each 0.5 ft of penetration. A fairly constant rate of penetration

(approximately 0.16 ft per sec) was obtained by using a constant-flow valve in the hydraulic system of one of the two drill rigs used. The rate of penetration was manually controlled on the other drill rig to as near a value of 0.16 ft per sec as possible. The force required for penetration has been plotted for all samples on the detailed boring logs which will be discussed later.

Piezometers

20. Piezometers were installed in the clean sand formation on ranges 3-U, 10-D, 20-D and 34-D, as shown on plate 23. Most of these piezometers were installed during the second week of July.

Laboratory Investigations

21. The undisturbed samples tested in the laboratory included some sand samples and all samples of overburden soils, the general type samples, and the material taken from the density samples. Strength, sensitivity to remolding, and relative consistency values were determined for selected samples of the overburden materials. The remainder of the tube samples were sliced and inspected for disturbance and stratification. All samples were visually classified and, in cases where there was any doubt as to the exact type of soil, classification tests were performed. Some samples of cohesionless materials were selected for triaxial tests.

Classification

22. The soils were classified by the Corps of Engineers Uniform Soils Classification. This classification system is based on the

Casagrande Classification and is described in a subject pamphlet transmitted by letter, ENGWE 461, dated 8 February 1949, to all Corps of Engineers Division Offices. The discrepancy occurring in that classification in the differentiation between "silt" and "clay" has been avoided in this instance by using the plasticity chart letter symbol, ML, and the name "sandy silt" or "silt" for all soils which fall below the "A-line" and have liquid limits less than 50 per cent. This was done because the soils that fell into this category appeared to be more characteristic of silt than of clay.

23. Almost all of the sands are poorly graded (SP) and the greater part of them fall into the "fine" group. The description accompanying each boring log generally shows an abrupt change in the gradation of the sand from "fine" to "coarse, with gravel" at an approximate depth of 100 to 120 ft in each boring. Actually, the transition is not so abrupt as the description would indicate; the clean sands are generally fine immediately beneath the overburden soils and gradually become coarser with depth, with the gravel sizes becoming increasingly predominant at greater depths.

Density data

24. The dry densities of selected samples were determined by removing the soil from the tube in 2-1/2-in. increments with a cup auger. The increments that appeared to have come from the same stratum were combined after the natural density had been determined for each and a maximum and minimum laboratory density were determined. The minimum density was determined by pouring the dry sand into a 2-in. by 4-in.

cylindrical mold from a constant height. The maximum density was determined by compacting the dry sand in four equal layers with 25 blows of a 4-lb hammer falling 12 in. in the same mold. The relative density was then computed as follows:

$$RD\% = \frac{e_L - e_N}{e_L - e_D} \times 100$$

Where e_L, e_N, and e_D, are void ratios for the loose, natural, and dense strata respectively. It is well to remember that the values upon which relative density is based are the maximum and minimum densities which were obtained with a dry sand in the laboratory. They do not necessarily correspond to the maximum and minimum densities which exist in nature. The relative densities and natural densities have been plotted on the detailed boring logs and are discussed in detail later.

Strength determinations — overburden

25. Strength and sensitivity were determined from the results of unconfined compression tests on undisturbed and remolded samples. The unconfined compressive strength has been shown as cohesion, C_u, in this report, C_u being equal to one-half the unconfined compressive strength as measured by the total load at failure. The degree of sensitivity, S, is expressed by the ratio between the cohesion C_u in the unconfined, undisturbed state and the cohesion C_m in the unconfined remolded test (1)*. The liquid and plastic limits and natural water content were also

* Numbers in parentheses refer to references listed in the Bibliography.

determined for these samples. The relative consistency C_r was determined by means of the following equation:

$$C_r = \frac{\text{liquid limit - natural water content}}{\text{plasticity index}}$$

Routine testing procedures were used for all of these tests.

Photographic record of samples

26. Photographs were taken of all sliced samples to provide a record of the stratification and any disturbance of the samples that was apparent visually. Photomicrographs were taken of a large number of samples to detect any variation of grain shape within the immediate failure area. The photomicrographs were taken on a breakdown of sieve sizes for each of the various samples to detect any variation of grain shape within the sample.

Triaxial compression tests on sands

27. A limited number of triaxial compression tests were performed on undisturbed sand samples which could be removed from the tubes without excessive disturbance. The specimens were first tested by a constant-volume, variable-pore-pressure type of test, with chamber pressure, σ_3, equal to 55 psi and an initial pore pressure, U, equal to 45 psi. The effective lateral pressure σ_3', was then equal to 10 psi. After the test had been performed on the naturally stratified material, each specimen was mixed and remolded to its initial density and the test repeated.

Results of Field and Laboratory Investigations

28. A new method of field exploration was used in this

investigation which permitted the accumulation of test data on the soils in a manner not previously feasible. Therefore, a report of the test results in considerable detail is warranted. However, such detail is not desirable in the text of a general report and these data are included herewith as an appendix. A summary of the results is given in the next Part and reference can be made to the appendix for information on individual borings and other data.

29. The results of the laboratory tests are not presented as a separate Part, but are included where applicable in the discussion of various phases of the investigation.

PART III: RESULTS OF FIELD INVESTIGATION

Soil Conditions on Range R-2

30. The soil profile on range 2-D is shown on plate 24. The overburden soils were about 40 ft thick on the landward end of the plotted section and only about 30 ft thick on the riverward end. The overburden soils were highly stratified, consisting of alternate strata of silts, silty-sands, and some clays.

31. The deep sand formation was composed of two general groups; a series of clean but stratified fine sands that became coarser with depth, which was underlain by a series of fine to coarse sands which contained gravel, very scattered clay lumps, and occasional thin strata of silts and clays. As can be seen on plate 24, the thickness of the clean fine sands and elevations, top and bottom, of this upper series were variable. The upper series was thickest at boring R2-3, thinnest at boring R2-4, and deepest at borings R2-3 and R2-1. No thick deposits of sands having low relative densities were encountered, but thin strata at low relative densities were encountered in all borings. The relative densities have been shown on plate 24 (expressed to the nearest 25%) as 25, 50, 75, or 100 per cent, depending upon the average value of relative density of all specimens of each sample. The least dense samples occurred in borings R2-1 and R2-2, the most dense in boring R2-3. Penetration of the sampler varied from 2 to 2.5 ft and the penetration resistance was generally less than 8 kips in the upper series. Lignite was encountered in all borings and was present in such large quantities in some samples that dry densities of less than 80 lb per

cu ft were obtained. The lignite was encountered most frequently and in the largest quantities in borings R2-1 and R2-2.

32. The sand and gravel series below the fine sands was not completely penetrated in any boring. Only two samples were obtained, both in boring R2-4, which could be used for relative density determinations. The relative densities of the two averaged about 75 per cent. In boring R2-1, a penetration of the sampler greater than 1.0 ft could seldom be made as the hydraulic unit of the drilling rig was not operating at full capacity. The fact that the drives became shorter using this reduced maximum capacity may indicate that penetration resistance was increasing with depth.

33. No piezometers were located in this area. The water table as observed in each boring was 10 to 12 ft above the river as shown on plate 24. This water surface could be either a perched water table in the shallow sands or a true water table indicating water pressures existing in the deeper sands. It is believed that the water table observed in the borings was, in most cases, a perched water table since the piezometers have not given any indication of heads in the sand appreciably greater than the elevation of the river. ground water table,

34. The initial failure area was subsequently enlarged by successive smaller failures which included the location of borings R2-2 and R2-3. Thus, the samples from these borings were from soils that were definitely involved in a failure.

Soil Conditions on Range 1-U

35. The soil profile on range R1-U is shown on plate 25. The

overburden soils were 30 to 40 ft thick except in boring R1U-1 which was located in soils identified as a swale filling. The overburden soils were highly stratified, consisting of the usual silts, silty-sands, and some clays. The silts ranged from firm to soft, the silty-sands were at low relative densities, the clays were generally soft. Soils in the swale filling consisted primarily of firm clays and silty-sands.

36. The upper sand series was of extremely variable thickness, being thickest in boring R1U-2 and thinnest in boring R1U-1. No thick strata of material at low relative densities were encountered. The majority of the samples fell into the intermediate relative density bracket of 50-75 per cent. Thin strata of sand at relative densities of less than 50 per cent were encountered in all borings but occurred most frequently in borings R1U-2 and R1U-4. Penetration of the sampler was generally less than 2.5 ft but greater than 2 ft with a penetration resistance of 6 $\pm$ 2 kips. Lignite was present in the sands in borings R1U-2 and R1U-3.

37. The lower sand series was highest (elev -25 to -30 ft) in borings R1U-1 and R1U-4 and lowest (elev -50 ft msl) in borings R1U-2 and R1U-3. The relative densities, penetration of sampler, and penetration resistance did not vary significantly from the upper series.

38. Piezometers were not located in this area. The water table in the clean sands, as determined from the boring logs, was 10 ft above the river.

Soil Conditions Parallel to the River

39. The soil profile at the top of the bank, parallel to the

river, from range 9-U to range 34-D is shown on plate 26. The varying depth of the overburden between ranges 1-U (shown on plate 23) and 34-D is probably due to an alternating ridge and swale type of accretion topography. The overburden is shallowest at ranges 34-D, 20-D, and between ranges 2-D and 1-U, indicating perhaps that those borings are located in sand ridges. Borings R9U-1 and R3U-1 are definitely located in the swale filling. Since the profile on plate 26 is diagonal to the direction of these features, there is a tendency for the width to be exaggerated.

40. The upper sand series is thinnest immediately behind the swale filling and thickest on ranges 1-U and 34-D. Consistently high relative densities, averaging about 75 per cent, were obtained in this series on ranges 9-U, 3-U, 10-D, and 30-D. Only a limited number of samples were obtained on range 10-D. The lowest relative densities were obtained on ranges 1-U, 2-D, 20-D and 34-D. In general, the length of drive possible to attain was least, and/or penetration resistance was highest on the ranges where the consistently high relative densities were obtained.

41. The relative densities, length of drive, and penetration resistance did not vary significantly in the lower sand series as compared with the upper sand series. The average elevation of the top of the lower series is at about -40 ft msl.

42. The indicated water table varies from 1 ft below the river at range 34-D to 12 ft above the river at range 10-D to 6 ft above the river at range 9-U. Piezometers were located on ranges 34-D, 20-D, 10-D, and 3-U. The water tables, as indicated in the boring logs, do not agree with the piezometer data except on range 34-D. Piezometer readings were

not started until after the borings had been made, but there was never any indication that a river-piezometer differential greater than 2 ft existed in the area. The water table in the boring may be influenced by a local perched water table in the overburden soils.

Backswamp Soils

43. The combined results of borings R2U-1 and R2U-2 plotted on plates A13 and A14, respectively, of the appendix, indicate that the backswamp soils occur behind the levee to a depth of 80 ft (elev -25 ft msl). The fragment of cohesive soils encountered from a depth of 70 to 80 ft in boring R2U-1 probably represents the toe of the backswamp deposit, the limit of past river migration and, therefore, the approximate limit of point bar soils in the area. The backswamp soils consist primarily of lean to heavy clays having fairly high strengths as compared with the point bar overburden.

44. Sands, corresponding to the upper sand series, as previously defined, are practically nonexistent beneath the backswamp clays. Scattered gravel of the lower series was encountered 10 ft below the backswamp clays in boring R2U-1 and 20 ft below the clay in boring R2U-2. The relative densities of the deep sands were seldom less than 50 per cent and commonly were between 75 and 100 per cent. The indicated water table in these borings is at approximately elev +50 msl or about 10 ft above the river at the time the borings were made.

Summary

45. A deep swale filling, consisting of firm clays and silty

sands, exists to an elevation of approximately -10 ft msl along the bank upstream from the failure area, at least as far as range 9-U (plate 23). It emerges from the bank at about range 0 and, based on the survey of 14-24 June, extends outward in a downstream direction to about range 4-D where it has been eroded to an elevation of about +20 ft msl. This deposit is underlain by approximately 30 ft of clean, fine sand that is probably more dense than similar sands in adjacent areas. The fine sand is underlain by a deposit of coarse sand and gravel which extends to elev -100 ft msl according to Fisk (2). The combination of the depth of the swale filling and the density of the sand have contributed to the resistant character of the bank in this area.

46. Typical silty point-bar overburden soils exist over the remainder of the area along the river bank and as far back as the toe of the levee. These soils are thinnest, existing to an elevation of only +20 to +30 ft msl, immediately adjacent to the swale filling on ranges 1-U and 2-D, and in local areas on ranges 20-D and 34-D. The overburden soils are underlain by the upper series of fine, clean sands to an elevation of approximately -40 ft msl. Generally, the sands are poorly graded and are made up principally of sub-rounded grains as shown by plates 27 to 30. Compared with the sands in other areas, they are relatively less dense and contain larger quantities of lignite. The ground-water pressure in the entire series may have been equal to a head of water 10 to 12 ft above the river level upstream of range 20-D as indicated by the boring logs. The fine sand is underlain by a series of coarse sand and gravel existing to a depth of approximately -100 ft msl.

47. The overburden soils on the landside of the levee consists of

comparatively firm backswamp clays to an elevation of approximately -30 ft msl. A thin (10 to 15 ft) stratum of clean sand was found in this area and is relatively more dense than the upper series of clean sand in the point-bar area. These sands are underlain by a deposit of coarse sand and gravel of considerable thickness. The combined strength and thickness of the backswamp clays and the relative thinness of the underlying sands would probably combine to make the soils in this area relatively resistant to river action, should the river ever be located adjacent to them.

48. Plate 31 shows the location of the swale filling and backswamp deposit as determined from the boring logs. On this plate the downstream end of the swale filling is shown at about range 5-D but there is little doubt that the "points" shown by the 20-ft contour line from several different surveys represent the position of the lower end of the filling on the date of the survey. Thus, the fact that a point persisted after several failures was due to the existence of this swale filling composed of firm clays and silty sands which have a greater resistance than the adjacent soils.

49. The approximate location of the backswamp soils is also shown on plate 31, and it should be noted that the upstream and downstream extent was not determined. The remainder of the soils in the area are bar accretion soils which usually show a definite ridge-and-swale topography. In this particular vicinity only one swale was positively identified but the so-called "sand ridges" are evident in the varying elevation of the top of the massive sands now covered by a later deposited overburden.

PART IV: ANALYSIS OF FAILURE

Local Conditions of Bank and River

50. The average slope of the bank between ranges 6-U and 45-D, prior to the failures, was 1 on 3.5. The steepest slope was 1 on 2.5 on range 3-U and the flattest slope was 1 on 4.5 on range 15-D. Portions of the bank having slopes steeper than 1 on 3 existed between ranges 4-U and 1-U; 7-D and 12-D; 29-D and 33-D; and on 41-D. Slopes flatter than 1 on 4 existed between ranges 13-D to 23-D and on 44-D. The failures included portions of the bank where the slopes were both above and below the average. Failures did not occur at the steepest portion of the bank.

51. The fact that the failures occurred only in local areas rather than over the entire reach indicates that certain conditions prevailed in these areas that were not common to the entire bank. These local conditions could have existed either in the river, in the soils composing the banks, or in both. A discussion of the local river conditions is beyond the scope of this report. However, the following conditions were observed and are of importance to any type of analysis concerning the stability of the slopes:

- a. The entire point was under general attack by the river.
- b. The only deepening of the thalweg of any consequence, prior to or after the failure, took place at the extreme upstream end of the revetment at about range 0.
- c. No marked turbulence was visible two days after the failure of 30 May 1949.
- d. The visible river turbulence changed from day to day in the weeks following the first failure, varying from extreme turbulence to practically none and vice versa.

the following paragraphs are devoted to a discussion of the stability of the bank on the basis of local soil conditions.

Possible Types and Causes of Failures

Characteristics of the failures

52. All of the failures had certain features in common, as can be seen on plates 32-38. They enlarged from a relatively narrow neck at the lower elevations to a wide, deep embayment near the top of the bank. The slope at the top of the bank was almost vertical down to an elevation of 0 ft msl, and then generally became very flat (5-10°) to the toe of the failure. The borings made in the river bed indicate that the bottom of the failure may have been 20 to 30 ft below the river bottom at the points where these borings were made and the soils forming the bottom in the vicinity of the borings were probably failed material from the top of the slope. The indicated toe of the failures, however, was seldom below an elevation of -40 ft msl, which suggests that the major portion of the failure took place in the upper series of the clean sand formation. The failure in the overburden soils was probably secondary to the failure in the clean sands and, apparently, the failure itself never extended into the lower sand series except to a very limited degree.

60' vertical face.

?

Contributing factors

53. The following factors may have contributed to the failure in some degree:

- a. Scour at the toe of the slope. Even though it is not indicated in the various surveys, some scour and minor slope readjustment may have occurred prior to all of the various failures. The scour may have been of large magnitude but

if so it must have occurred very rapidly and immediately before the failure since it was not detected in previous surveys. On the other hand, it may have been so slight that it was not detected, but still was capable of causing enough slope readjustment to affect the stability of the bank.

b. Rapid drawdown and bank seepage. Although the river was actually rising at the time the first failure occurred, it had been falling at the rate of 1 ft a day for several weeks previously and had fallen about 25 ft in the two months prior to the failure. This may represent a condition somewhere between complete, instantaneous drawdown and steady seepage.

c. Internal erosion. The stability of the bank might have been seriously altered by internal erosion. (A complete discussion of failures caused by this phenomenon can be found on pages 507-510 (1).) Internal erosion implies that seepage removes certain portions of the soil from the interior of a soil mass, either in channels or by leaching of the fines. Canalization is the most probable form of internal erosion; it occurrs in sands immediately beneath a stratum which is cohesive enough to maintain its structure over the channel. The channel grows larger and larger until the roofing soil finally falls, either by a sudden movement or by a general subsidence which may occur over a period of years.

d. Stratification. Careful inspection of undisturbed samples of sand showed that in the majority of cases the samples were highly stratified. Stratification apparent on plate 39 is due to differences in materials i.e., grain-size distribution, type of soil, etc., in some cases, and in other cases, to a difference in density in a single material. Many strata were less than an inch in thickness and strata were seldom encountered which could have had a thickness greater than 2-1/2 ft. Cross-bedding was quite common and on this account it is practically impossible to predict the continuity or areal extent of a thin stratum. Much has been learned as to the nature of the stratification of these sands but the effect of the stratification on the over-all stability of a river bank has not yet been determined.

e. Miscellaneous factors affecting the strength of the deep sands. The upper series of the clean sands may have been weaker in the failure area because of the lignite, lower relative densities, and lower confining pressures due to less overburden. They may also have been weaker in the sense that some inherent factor such as grain shape or structure makes these

sands susceptible to liquefaction or flow type of failure.

Shear failure

54. The static forces tending to cause failure in a bank are as follows: If the soils above the river elevation are not drained after drawdown, the saturated unit weight of the soils and seepage forces will constitute the driving force tending to cause failure. The driving force will be resisted by the combined strengths of the overburden, due to cohesion and confining pressure, and of the sands due to confining pressure alone. All other things being equal, shear failures will tend to occur where the slopes are steepest.

55. Since rapid drawdown is generally more critical than steady seepage for large failures of the type which occurred, slopes were analyzed on the assumption that complete, rapid drawdown occurred from the top of the bank to the river elevation (40 ft msl) at the time of the failure. The exact depth and extent of the failure are not known but since the static strength of sand increases with depth and confining pressure, the most critical surfaces are generally shallow. In the analyses made herein, the strengths assigned to the overburden were typical of the soils and corresponded most nearly to the results of laboratory strength tests made on samples from borings adjacent to the failures. The slopes were analyzed to determine what strengths were necessary in the sands to maintain a factor of safety of unity.

56. The size and shape of the failure at ranges 6-D to 16-D are shown on plate 32. Plates 33 to 35 are profiles of various sections and ranges for the before- and after-failure slopes. The profiles for the small, subsequent failure on ranges 13- and 14-D are presented on plate 36.

The flat slopes after failure are readily apparent. The bank remained generally unchanged at range 2-D (plate 33) and range 4-D (plate 34) immediately after the first failure. The exposed point caused by the swale filling extended through range 6-D, as evidenced by the peak in the 7 June survey shown on plate 34. Section AA, shown in plan on plate 32 and in section on plate 33 most nearly corresponds to the direction of the axis of the failure. A stability analysis of this section indicates that the most critical surface corresponded to the after-failure surface. The angle of internal friction of the deep sands needed only to have been 12.7° to maintain a factor of safety of 1. The analysis of the subsequent failure of the exposed point (assuming that it failed in a direction parallel to the range lines) indicates that an angle of ~~friction of 23.2°~~ would be sufficient to maintain stability. The sand in the small failure at range 13-D needed an angle of internal friction of only 12.9° to maintain stability.

what about before failure

57. The size and shape of the failure on ranges 16-D to 22-D are shown on plate 37 and profiles for ranges 19-D and 20-D on plate 38. The sand needed an angle of internal friction of only 11.5° to maintain stability on a surface corresponding to the after-failure survey on range 19-D. A deeper possible failure surface extending from the top of the bank to the thalweg (which includes the steeper slope immediately above the thalweg) was more critical. An angle of internal friction of 13.6° was needed to just maintain stability.

After failure?

58. Thus, according to conventional methods of analysis for simple shear failure, friction angles of from 12° to 23° are required for factors of safety of unity at the locations of the failures. Triaxial

compression tests made on selected undisturbed samples indicated the angle of internal friction to be about 35°. This, of course, indicates a large factor of safety for all bank sections analyzed according to conventional methods. The relative density of the samples used for the triaxial compression tests varied from 82-99 per cent which showed that these sands were in a dense state, probably due to jarring and shocks during shipment to the laboratory. The angle of internal friction would certainly have been less than 35° for these same sands in a medium and loose state of density and the indicated factor of safety would have been correspondingly lower, but still considerably higher than unity.

Flow failures

59. The strength of sand varies directly with the angle of internal friction and the intergranular pressure. Thus, with the angle of internal friction remaining constant, a reduction in strength accompanies a reduction in intergranular pressure. A failure occurs when the intergranular pressure is reduced to the point where the strength is less than the forces tending to cause movement. Such failures are easily demonstrated in the laboratory by the familiar "quicksand" device and by the triaxial compression test. The principles involved also apply to natural deposits of sand but due to the relatively great dimensions of sand masses in the field and to stratification, variations in density, etc., the direct application of laboratory test values to field conditions can only be made in a qualitative sense.

60. The pressure in the pore water of a mass of saturated sand is of primary importance because excess pore pressure results in a

lowered intergranular pressure and consequently lower strength. Excess pore pressures of a magnitude sufficient to cause complete liquefaction of laboratory specimens are readily developed in sands of low density. When strains occur in saturated sands of low density the sand grains tend to undergo a re-arrangement which produces a more compact mass (higher density), and a reduction in volume of the sand-water mass is required. The necessary volume decrease is accomplished by water draining from the mass. If conditions are such that drainage can occur freely (high permeability of the sand or low rate of strain) excess pore pressure may not reach a dangerous point. On the other hand, when drainage is prevented, pore pressure may easily reach a value that will quickly reduce intergranular pressures and cause partial or complete liquefaction of the mass. The results of other triaxial compression tests on sands of Mississippi River banks [3] illustrate the susceptibility of loose sands to liquefaction when in an undrained condition and also show that liquefaction is not likely to occur in sands of high density or in a completely drained condition. The same effects, but to a lesser degree, are shown by a limited number of tests made on undisturbed and remolded samples from the boring on range 20-D at Morville. The results of these tests are shown on plate 40. It can be seen that the minor principal stress, σ_3, is reduced initially in tests of both undisturbed and remolded specimens and that the reduction is greater for the undisturbed specimens. The greatest reduction occurs at a strain of about 1 per cent. These specimens were at a high relative density and the reduction would be much greater for samples at a lower relative density.

61. It has also been learned that partial or complete liquefaction

occurs in undrained laboratory specimens of loose sands at a strain of about 1 per cent. The fact that failures can occur at such small strain values was not formerly recognized and suggests interpretations that may be extended to the field. The possibility of flow failures occurring in river banks made up of sands has been given considerable attention in the past but it was generally believed that some shock or other "trigger action" was required to set off a flow failure. If small strains in natural deposits of loose sand cause excess pore pressures, these pressures may be of sufficient magnitude to initiate a flow failure, or to act in a contributory manner and cause relatively small simple shear slides to assume considerable magnitude due to the removal of soil and the consequent strain required for adjustment of the soil to the new stress conditions.

62. Forces acting within a river bank are changed by the rise and fall of the river, change of bank slopes due to scour, and possibly other factors, but the magnitude of the strain accompanying the changes in forces has not yet been determined. This problem is believed to be of considerable importance and to warrant further study.

63. Simple shear failures have been discussed earlier and it was concluded that the river bank at the site of the failures had an ample factor of safety as determined by conventional methods of analysis. It appears reasonably certain that the failures at Morville were not due solely to shear failures but it is possible that some combination or sequence of shear failures and partial or complete liquefaction may have occurred. So-called "flow failures" in extremely flat slopes have been reported[4] and there appears to be no doubt that some degree of

liquefaction is required. The exact mechanics of a flow slide and the sequence of events involved are not known. An entire sand mass flowing out as a true liquid would appear to be one extreme or limiting case. Another case might be a series of shear slides where the sand mass involved in each slide becomes more or less liquid during or after sliding and assumes the flat after-failure slope characteristic of flow failures. There are probably intermediate cases depending on the soil conditions and other factors at a particular site.

64. According to the best information available, submerged uniform fine sand with rounded to sub-rounded grains and at a relatively low density is highly susceptible to liquefaction and flow slides. Such sands existed at Morville; thus, a flow slide was possible. It cannot be positively established at this time that flow slides did occur but no other explanation can be offered to account for the relatively flat slopes in the failure area both before and after failure. It is believed that partial or complete liquefaction of portions or all of the sands involved in the failures must have occurred.

PART V: SUMMARY OF RESULTS

65. The following factual results of this investigation are considered to be of significance in defining the conditions found at the site and in drawing conclusions from this study.

a. The river bank at and in the immediate vicinity of Morville revetment is a point-bar accretionary deposit of massive sand formations overlain by a 30- to 90-ft overburden of slightly cohesive soils. One large swale filling was identified lying parallel to Natchez Island Chute and extending downstream to about range 0.

b. Failures occurred in fine sand and did not appear to extend into the deep-lying graveliferous sands.

c. Undisturbed sand samples showed the sand to be highly stratified and in many instances to contain appreciable quantities of lignite.

d. The sands were not well graded and were made up of subrounded to rounded grains.

e. Densities of the sands varied over wide limits. Strata of loose sands were found in every boring but, due to the complex stratification and cross-bedding, the continuity or areal extent of an individual stratum could not be established.

f. Pore pressures in the deep sands several weeks after failure were found to be too small to be of any practical significance. The water table, as indicated by the boring logs, was appreciably higher than the water surface of the river, but may be a perched water table or a temporary one caused by drilling operations.

Held up by asphalt revetment.

PART VI: CONCLUSIONS

66. Based on the data presented in this report and on information contained in the references cited, the following conclusions are believed warranted:

a. The failures at Morville revetment were not simple shear failures, as stability analyses indicate an ample factor of safety with respect to failures of this type.

b. The failures at the Morville revetment are believed to have resulted from partial or complete liquefaction of fine sand deposits. Factors which substantiate this conclusion are as follow:

(1) Conditions at Morville were favorable for the liquefaction or flow type of failure.

(2) Failures due to liquefaction have been reported in the field and the shape and description of the failure at Morville conform closely to those of known flow failures.

(3) Partial or complete liquefaction of loose fine sands has been demonstrated in the laboratory.[3]

(4) There is no other apparent explanation for the unusually flat slopes before and after failure.

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(3) Corps of Engineers, Waterways Experiment Station. Reid Bedford bend, Mississippi River, triaxial tests on sands. Potamology Investigations Report no. 5-3. Vicksburg, Mississippi, Waterways Experiment Station, 1950

(4) Koppejan, A. W. and others. "Coastal flow slides in the Dutch province of Zeeland." Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, vol 5, pp 89-96. Rotterdam, 1948.

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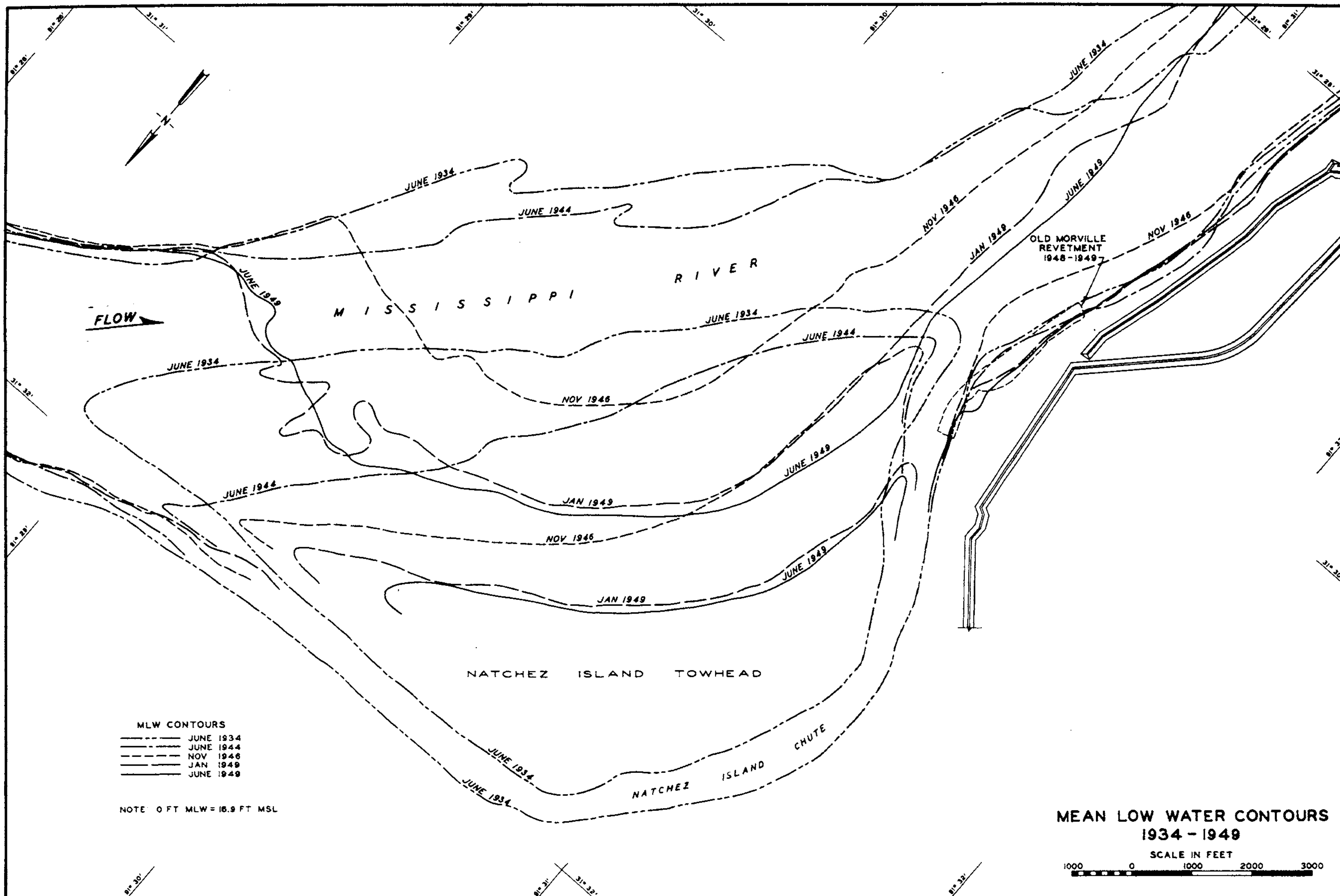
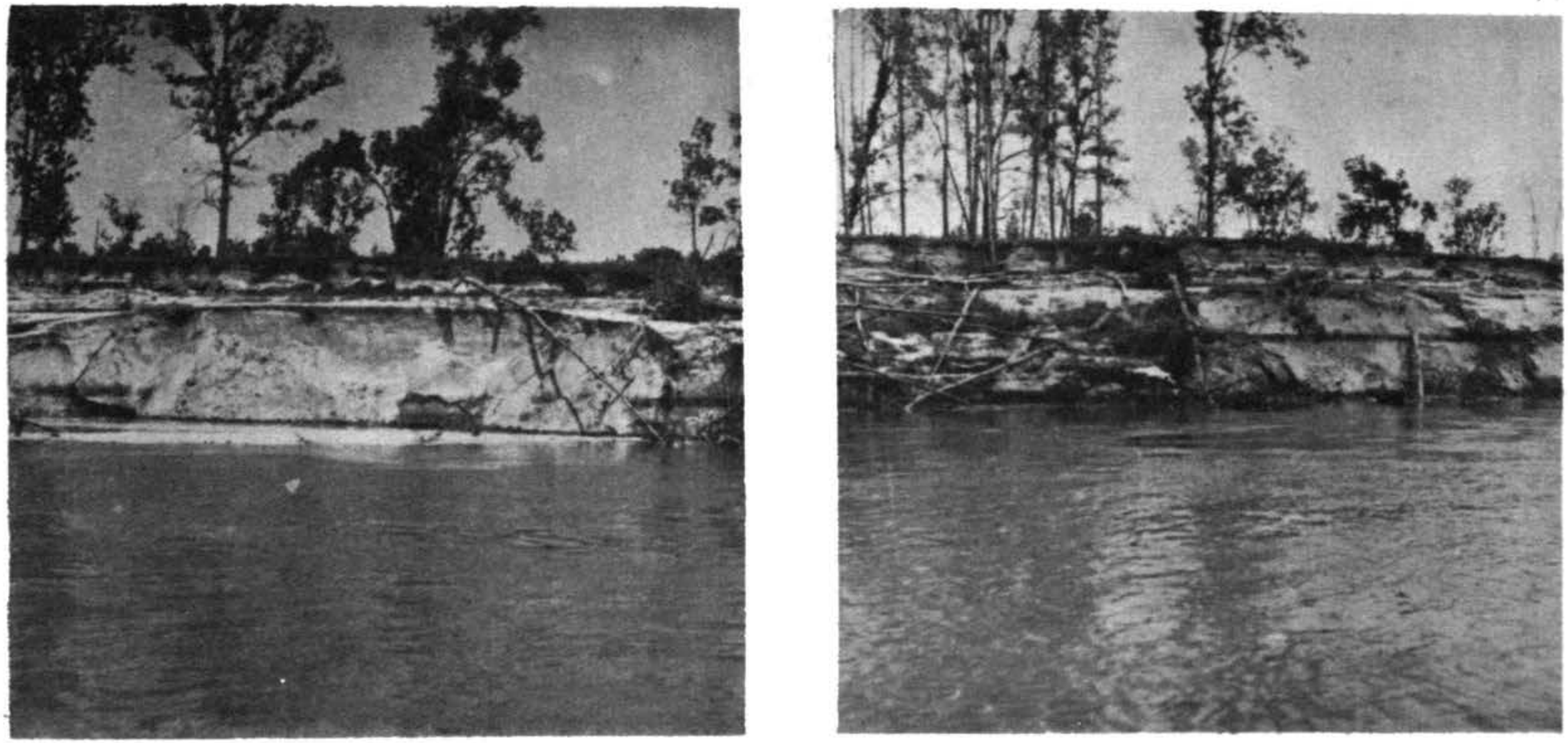
PLATES



N
FLOW
MISSISSIPPI RIVER
OLD MORVILLE REVETMENT 1948-1949
NATCHEZ ISLAND TOWHEAD
NATCHEZ ISLAND CHUTE
JUNE 1934
JUNE 1944
NOV 1946
JAN 1949
JUNE 1949
MLW CONTOURS
JUNE 1934
JUNE 1944
NOV 1946
JAN 1949
JUNE 1949
NOTE: 0 FT MLW = 16.9 FT MSL
MEAN LOW WATER CONTOURS
1934 - 1949
SCALE IN FEET
1000 0 1000 2000 3000

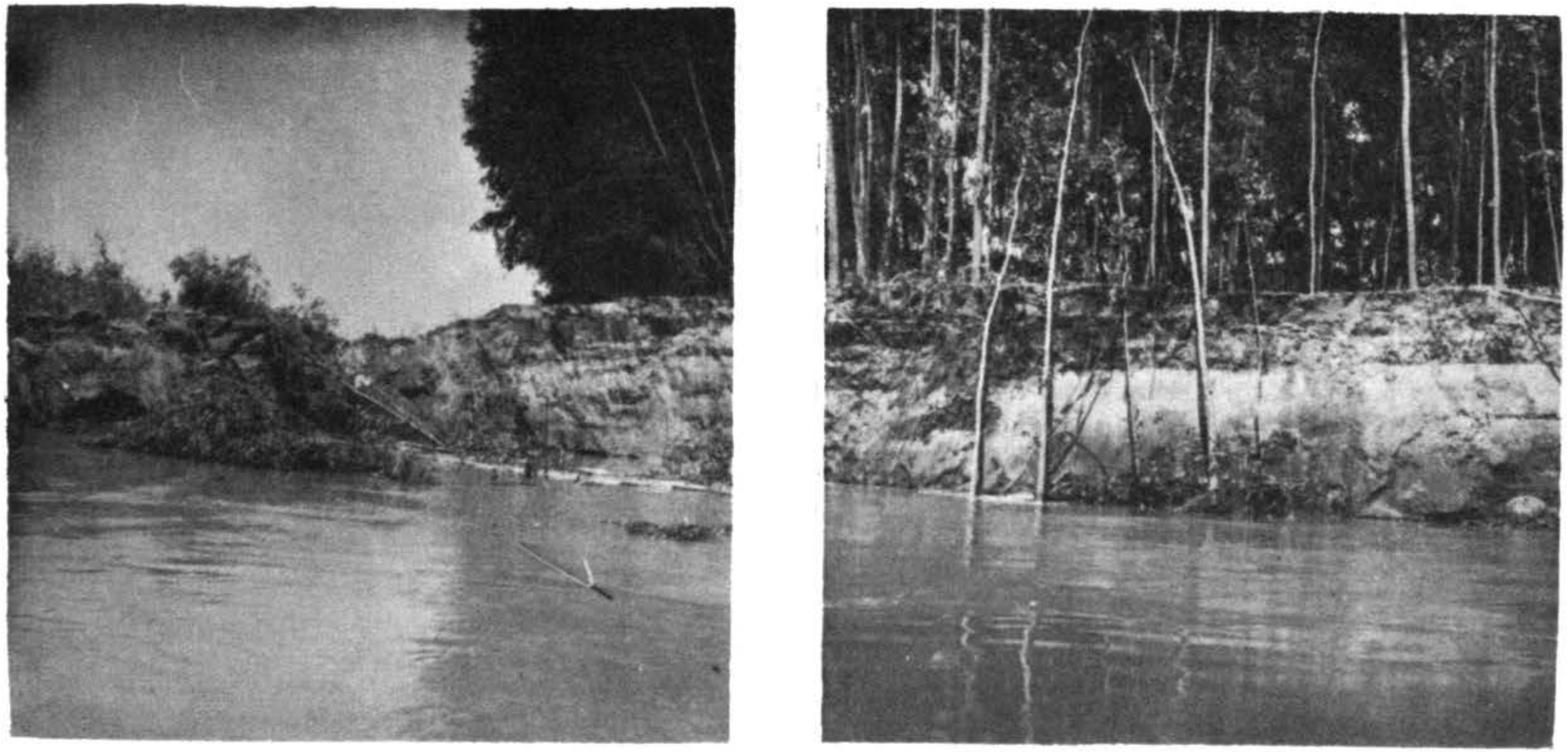
PLATE 1



BANK CONDITIONS ON NATCHEZ ISLAND

UPSTREAM POINT 18 JUNE 1949

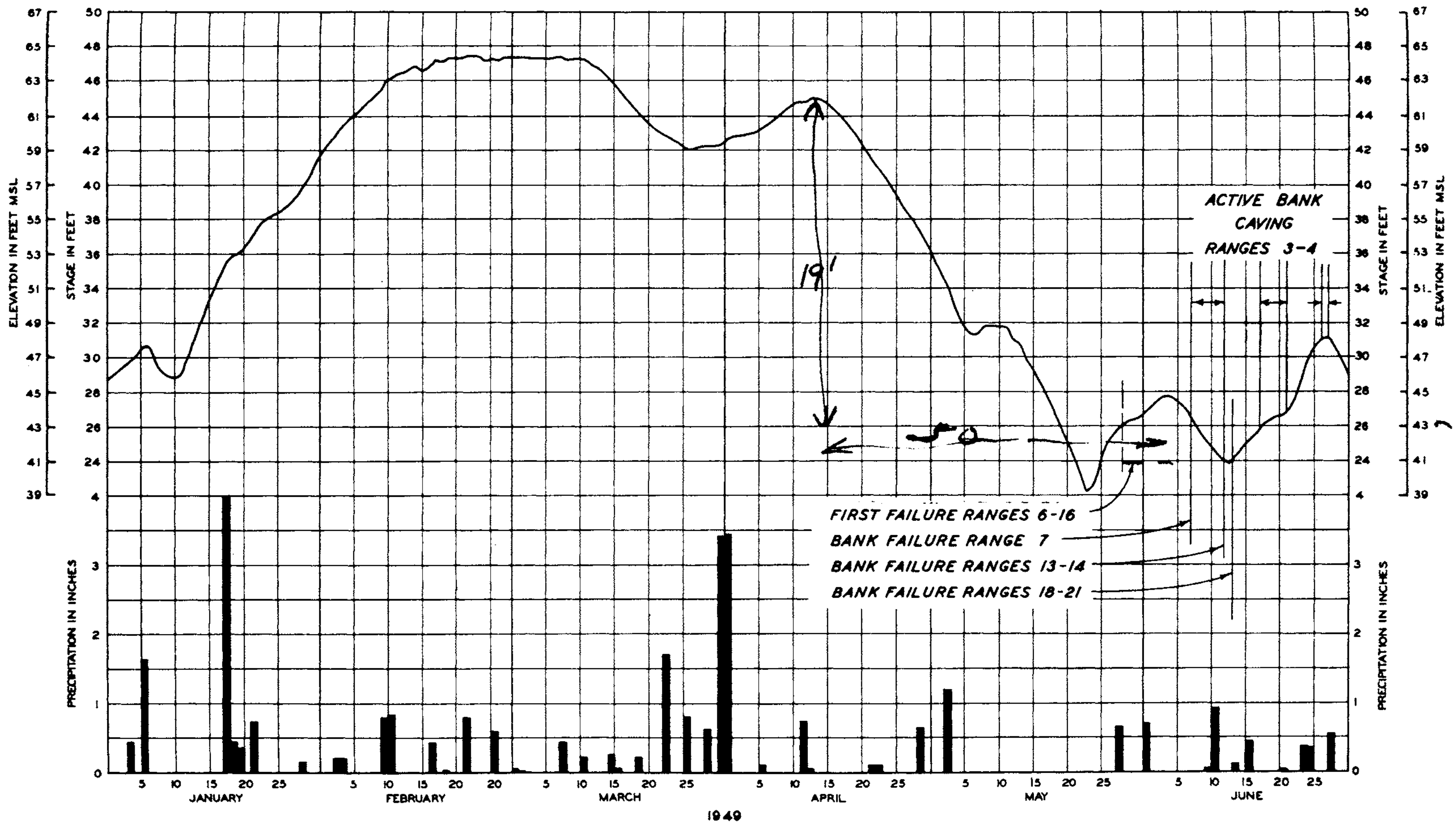
PLATE 2



BANK CONDITIONS ON NATCHEZ ISLAND

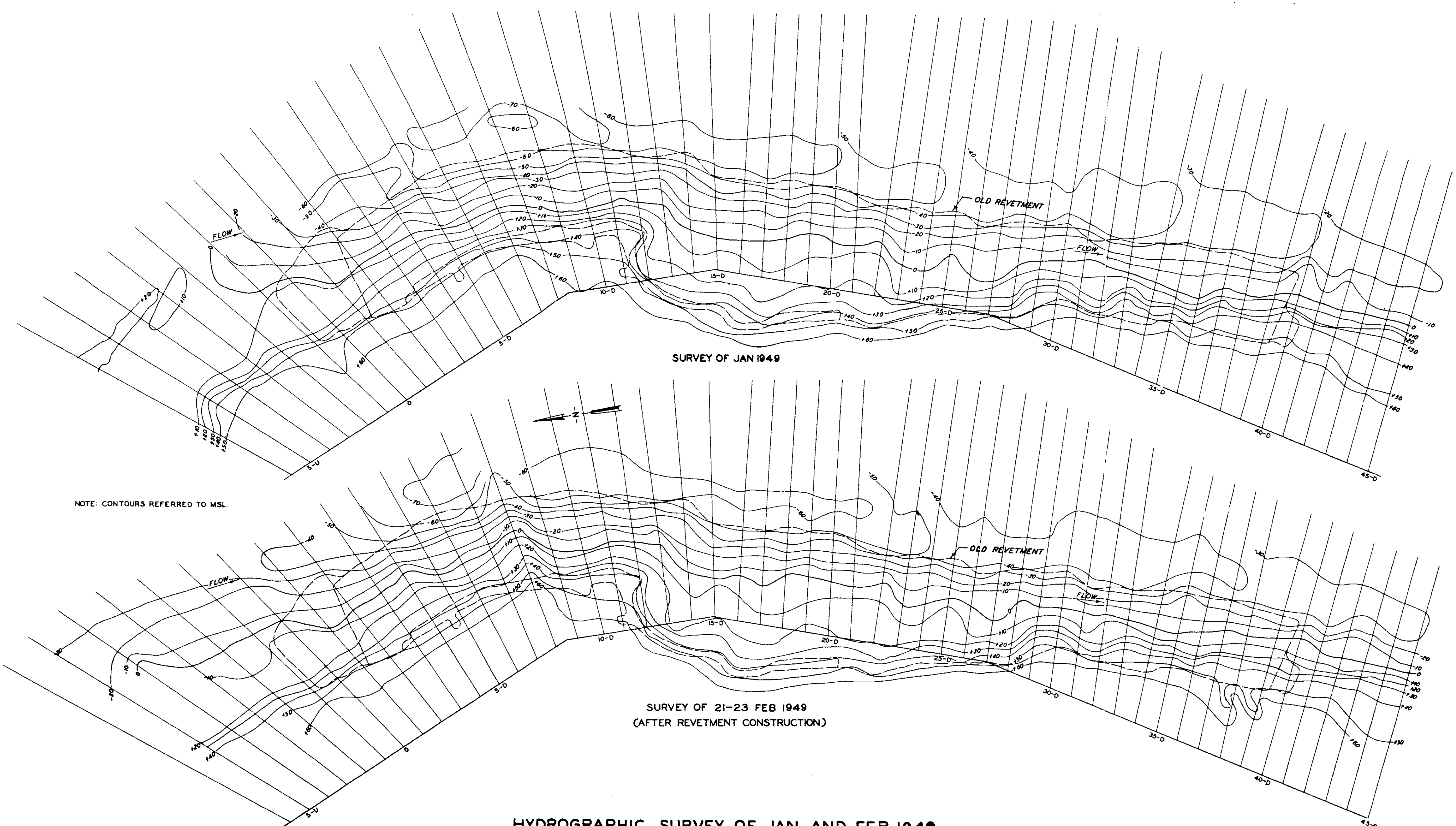
DOWNSTREAM POINT 18 JUNE 1949

PLATE 3

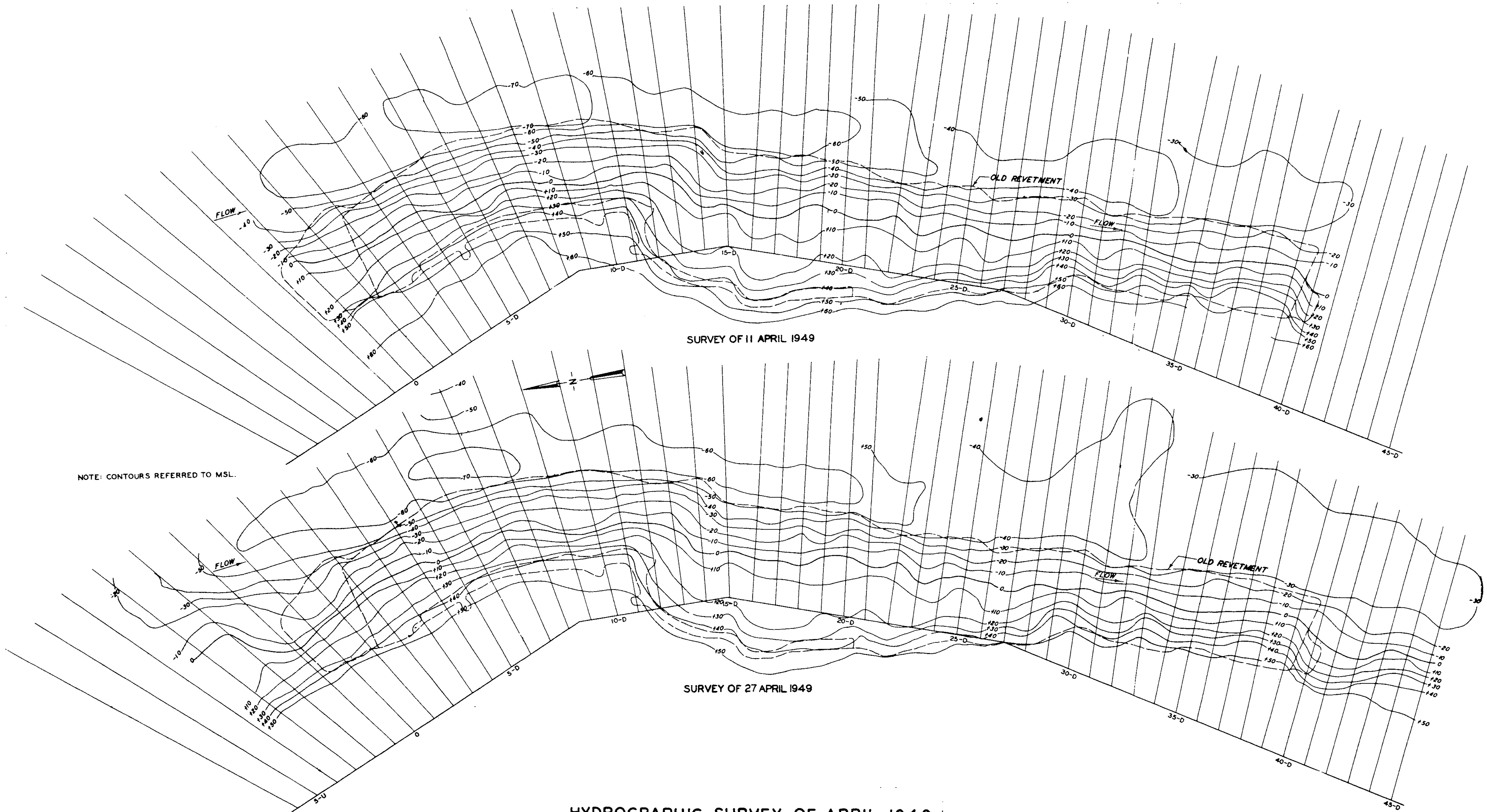


STAGE HYDROGRAPH
NATCHEZ BRIDGE GAGE

PLATE 4



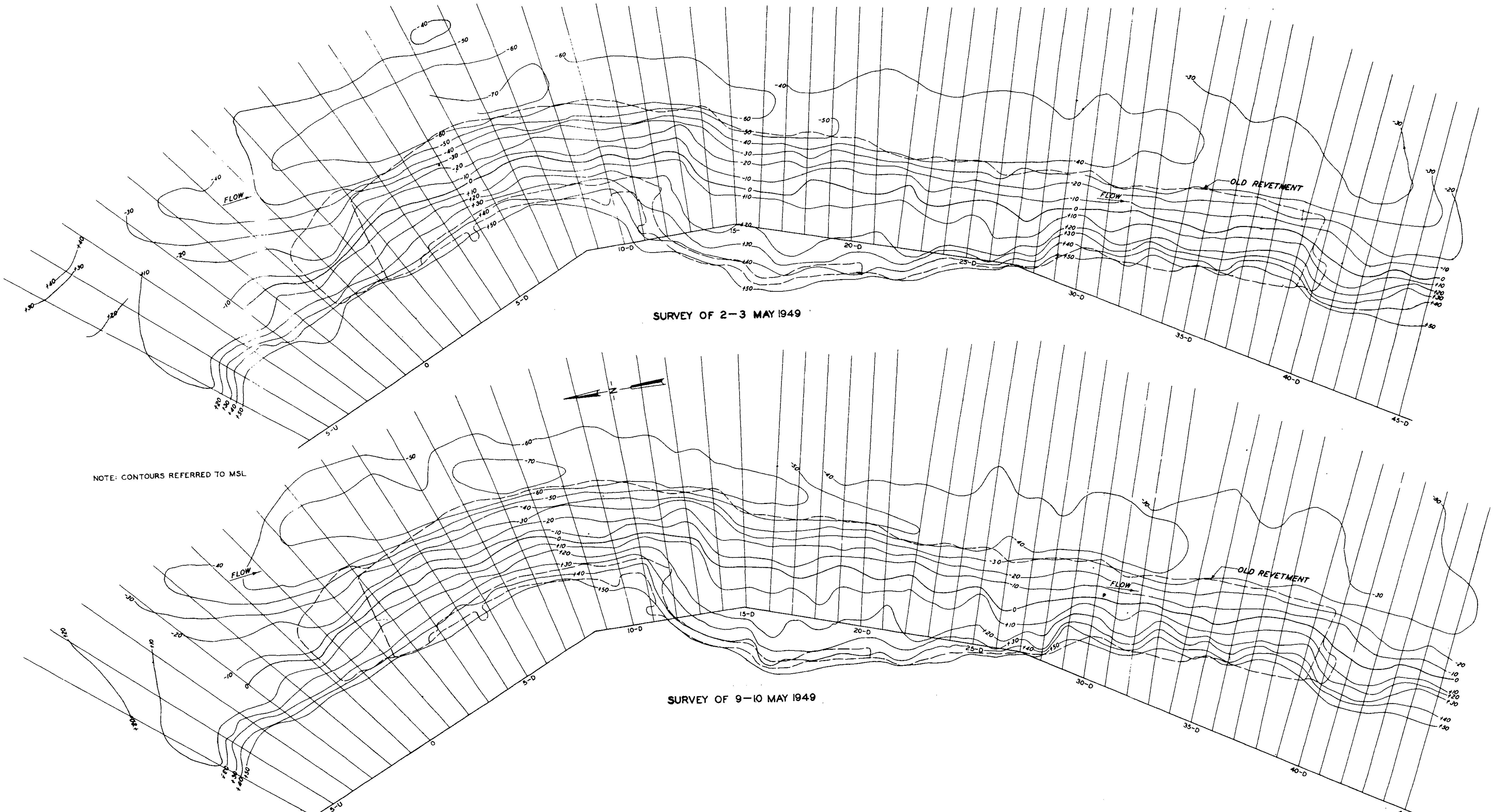
HYDROGRAPHIC SURVEY OF JAN AND FEB 1949



HYDROGRAPHIC SURVEY OF APRIL 1949

SCALE

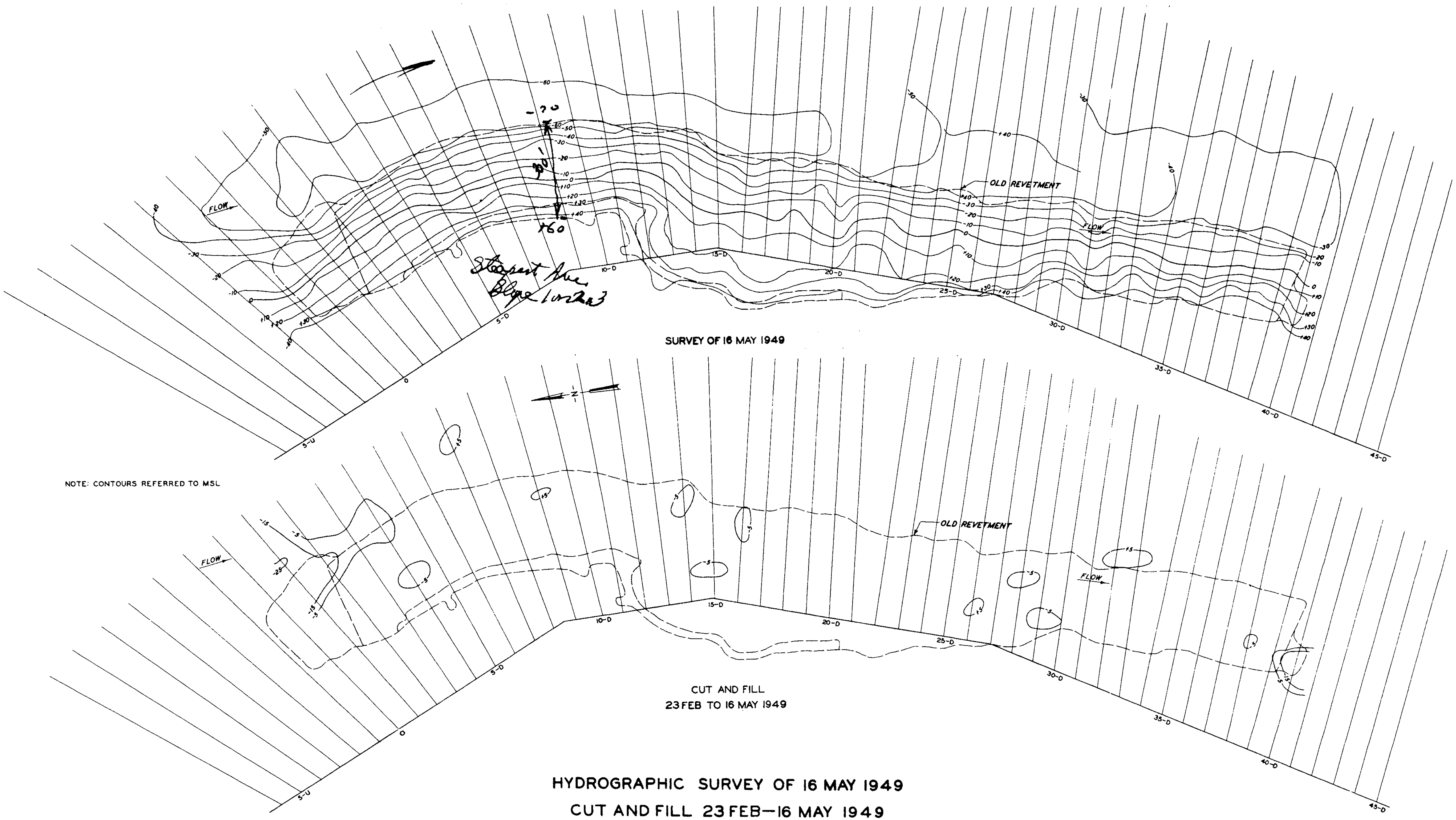
500 0 500 FT



HYDROGRAPHIC SURVEY OF MAY 1949

SCALE

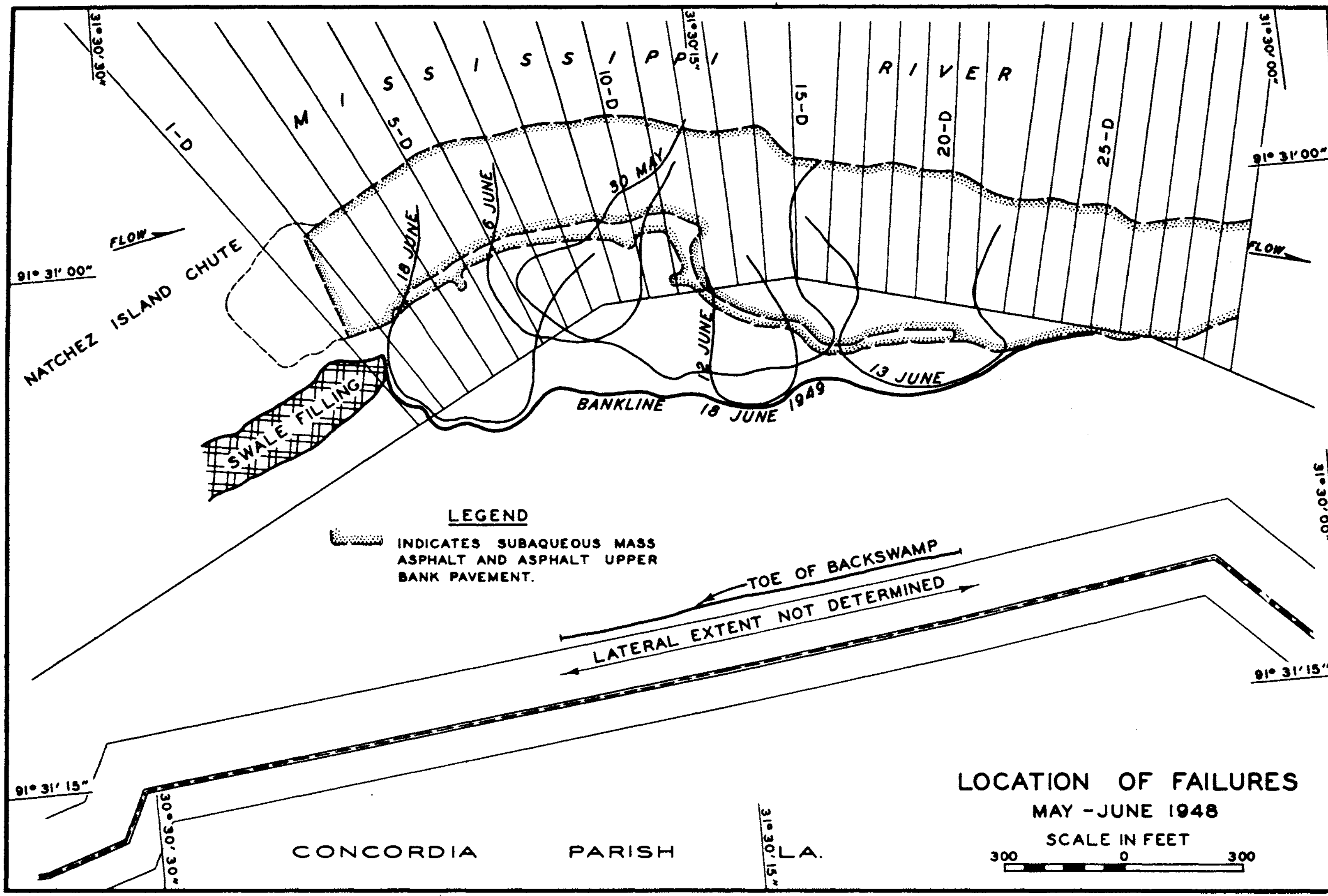
500 0 500 FT



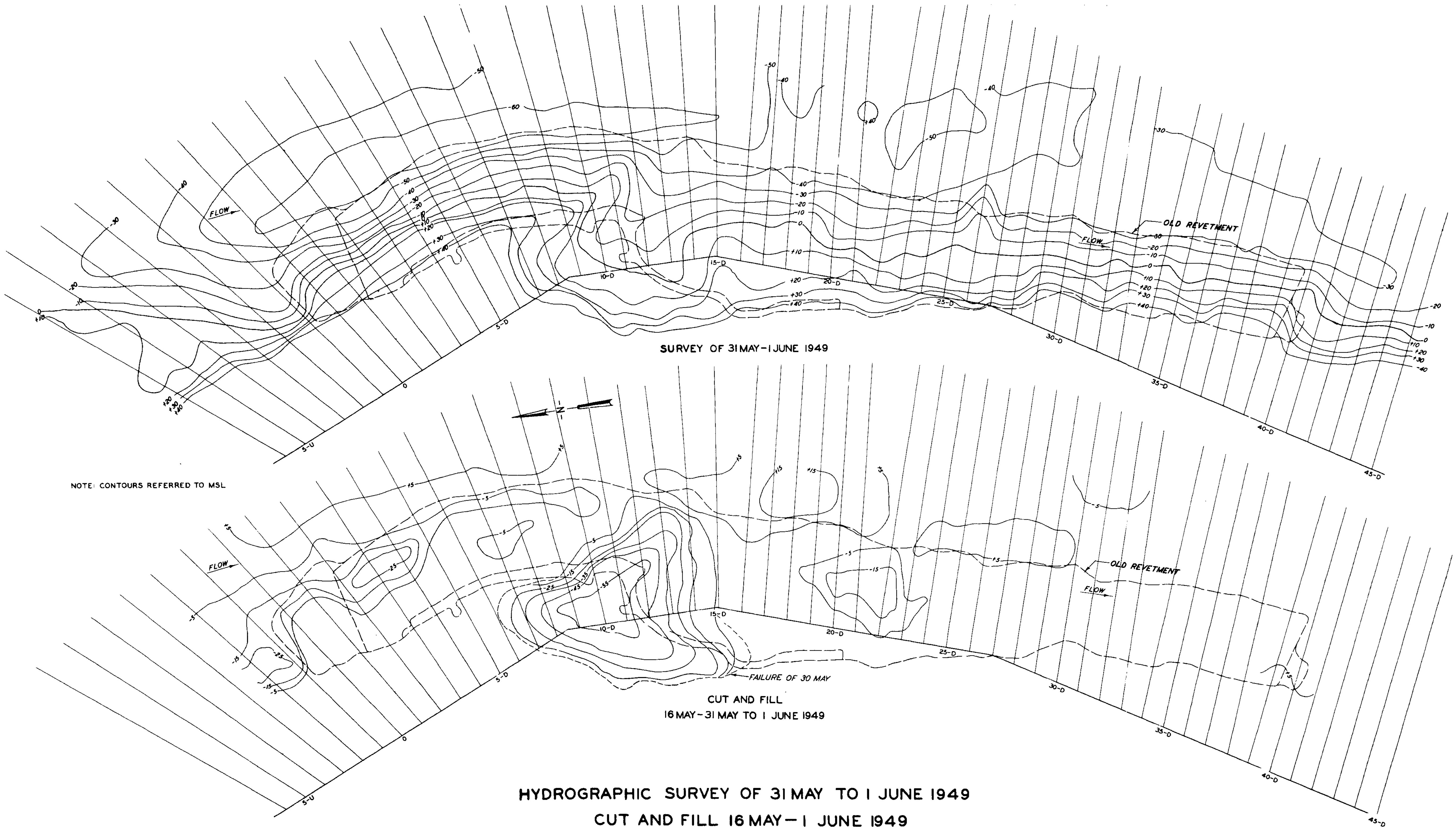
HYDROGRAPHIC SURVEY OF 16 MAY 1949
CUT AND FILL 23 FEB—16 MAY 1949

SCALE
500 0 500 FT

PLATE 9



LOCATION OF FAILURES
MAY - JUNE 1948



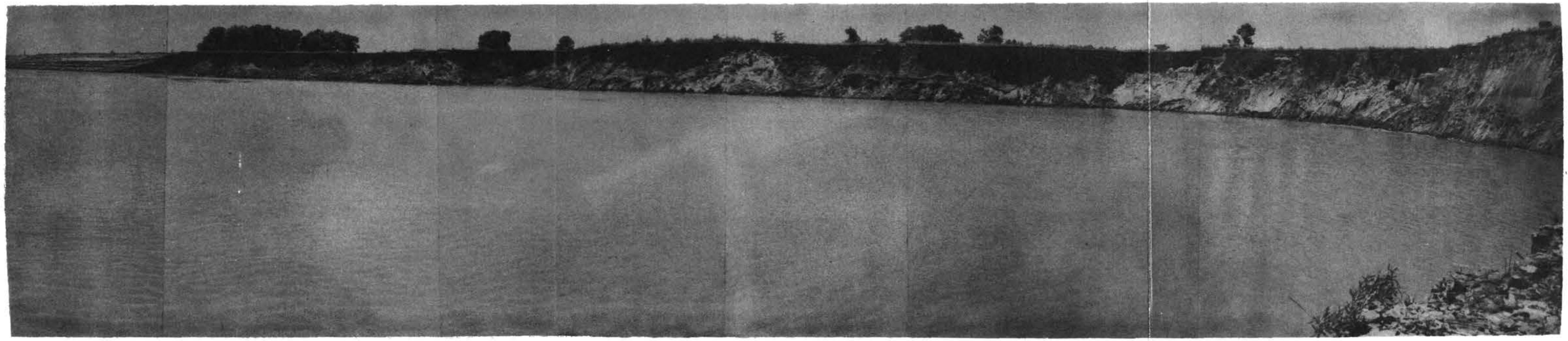
HYDROGRAPHIC SURVEY OF 31 MAY TO 1 JUNE 1949

CUT AND FILL 16 MAY — 1 JUNE 1949

SCALE

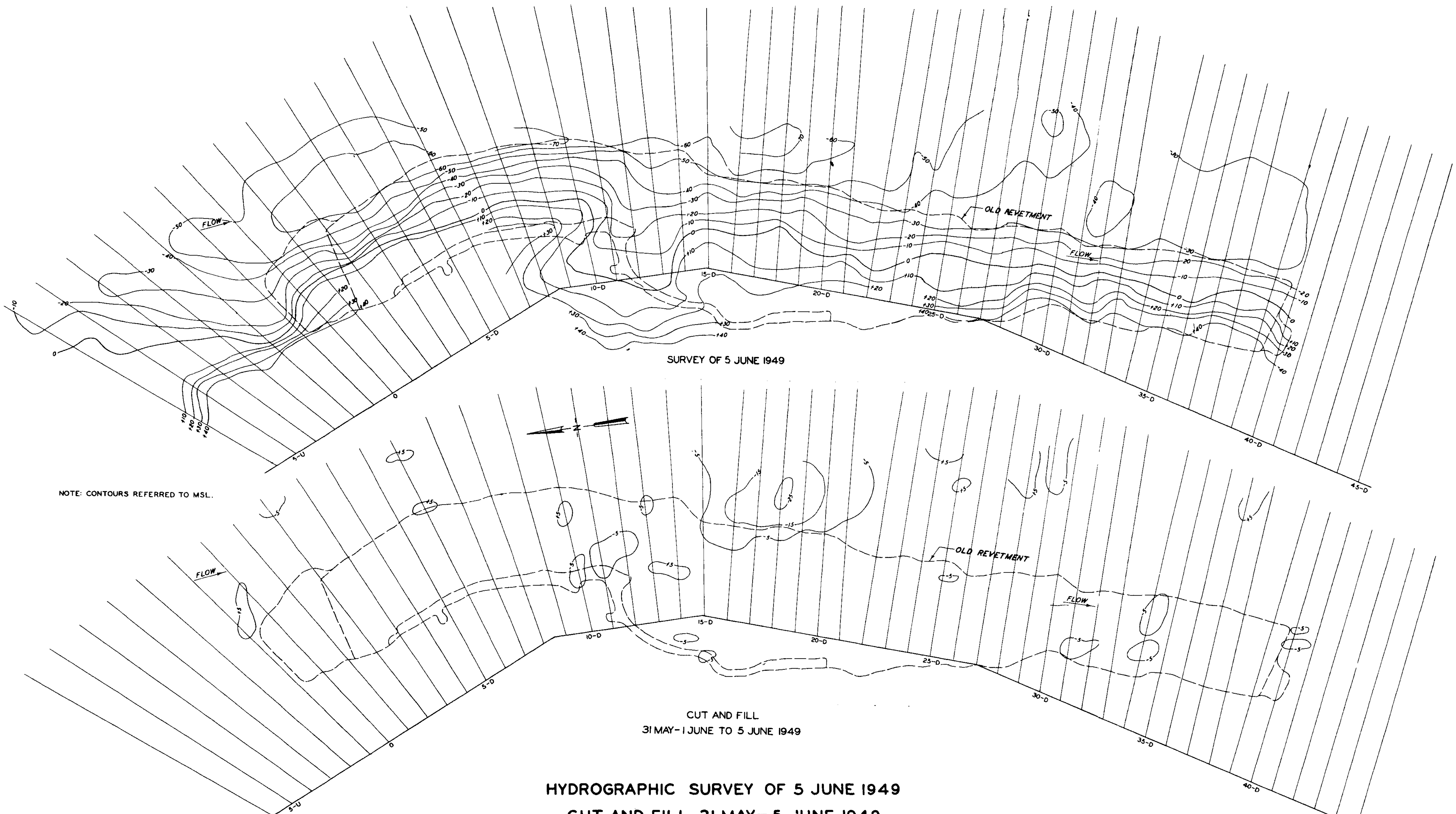
500 0 500 FT

PLATE 10



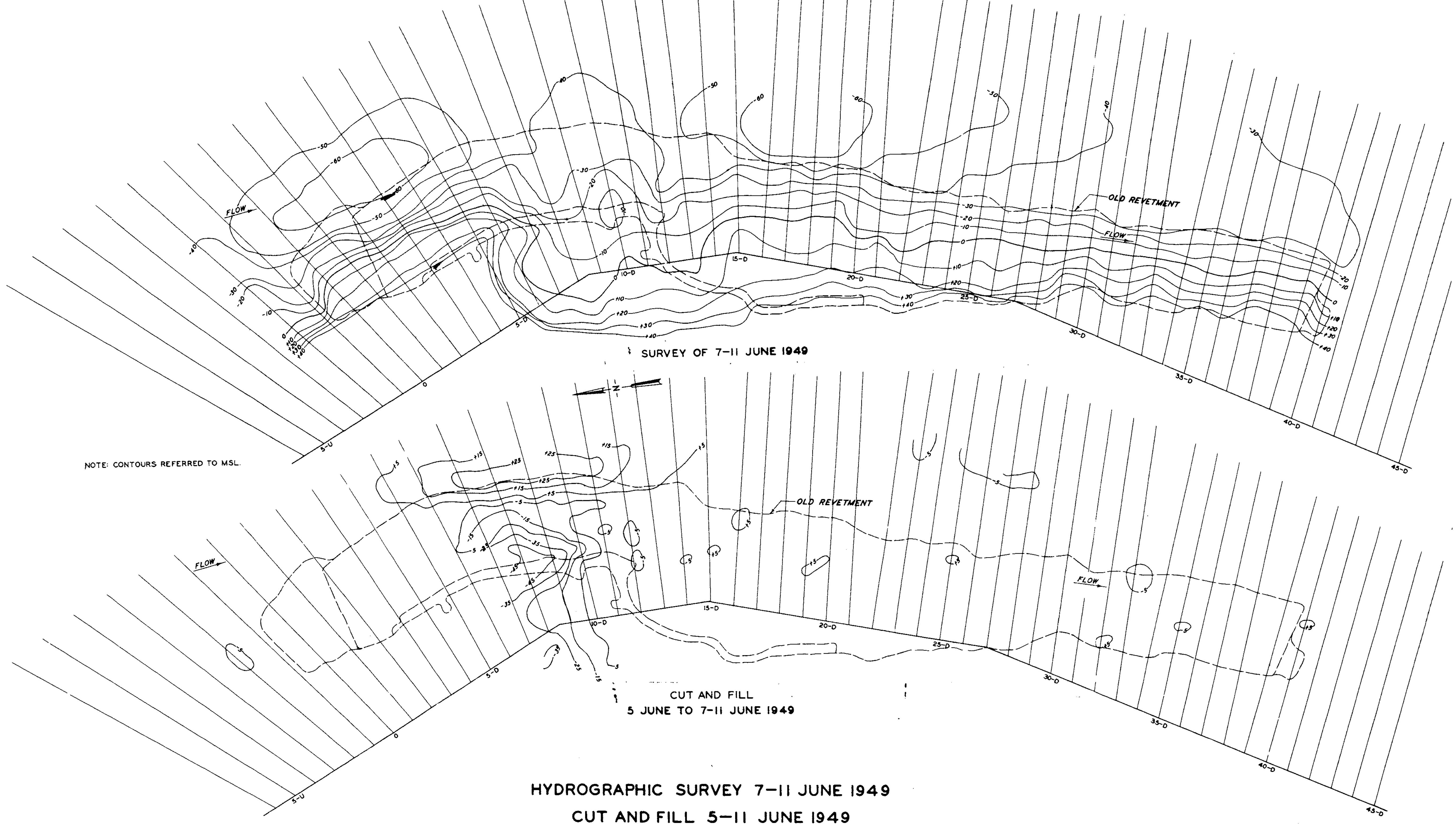
ORIGINAL FAILURE

TAKEN FROM POINT ON RANGE 7-D — 2 JUNE 1949



HYDROGRAPHIC SURVEY OF 5 JUNE 1949
CUT AND FILL 31 MAY—5 JUNE 1949

SCALE
500 0 500 FT



HYDROGRAPHIC SURVEY 7-11 JUNE 1949

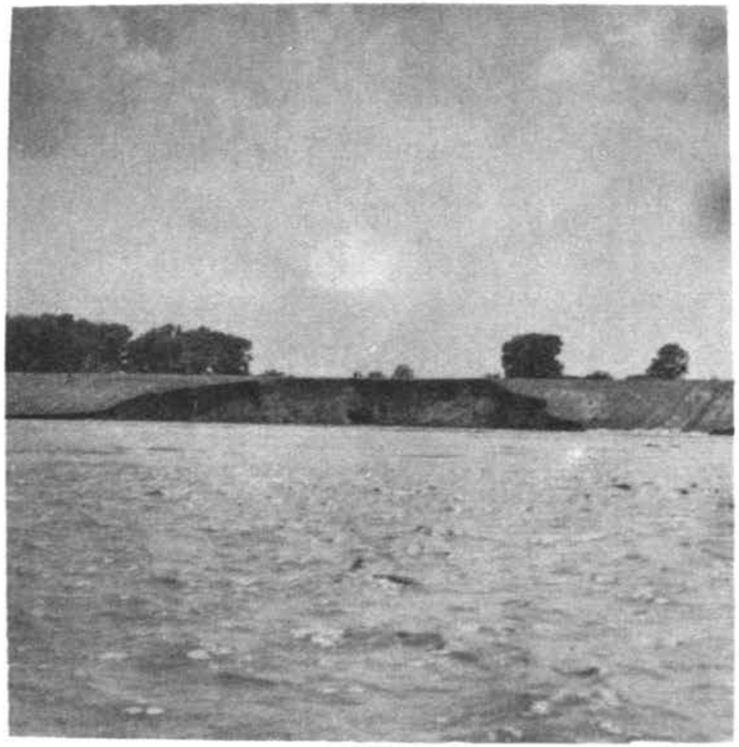
CUT AND FILL 5-11 JUNE 1949

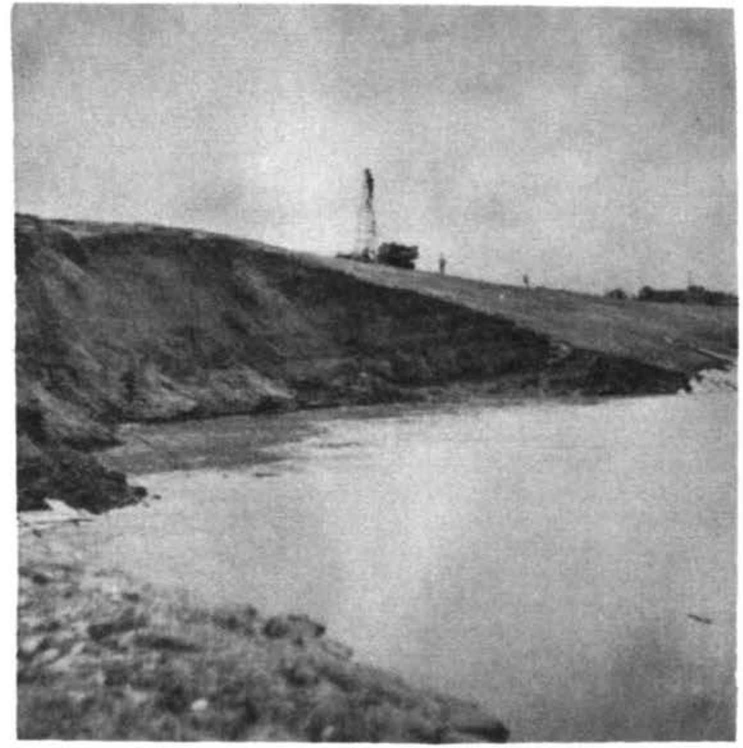
SCALE

500 0 500 FT

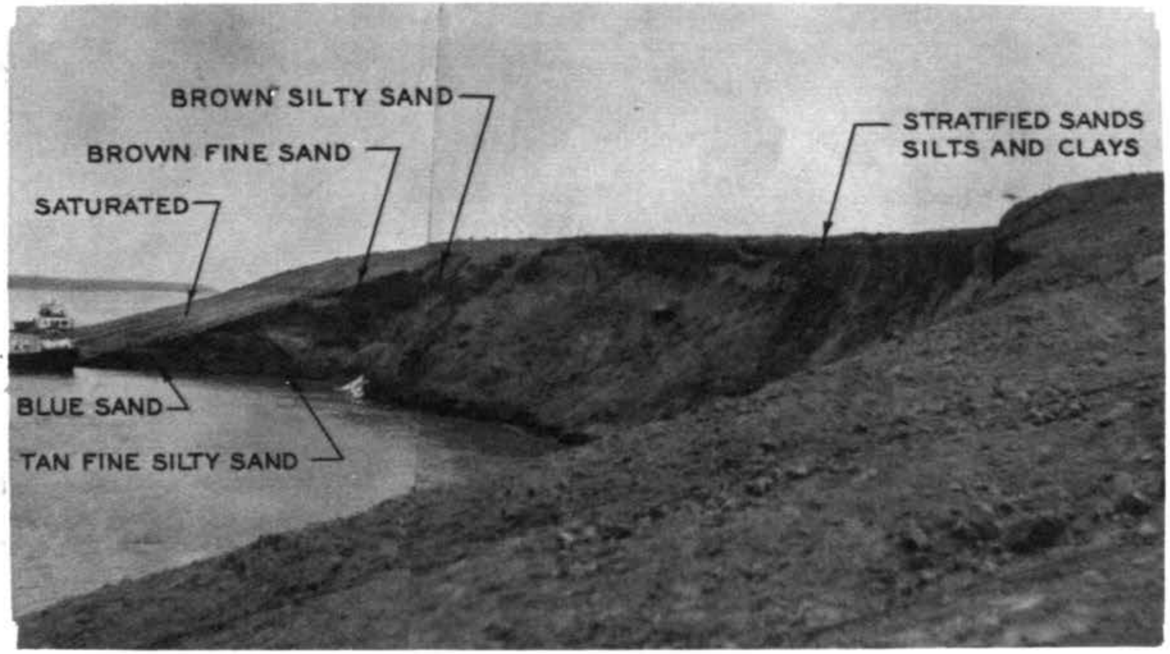
BANK SLUMPING AT RANGE 10-D
14 JUNE 1949

PLATE 14





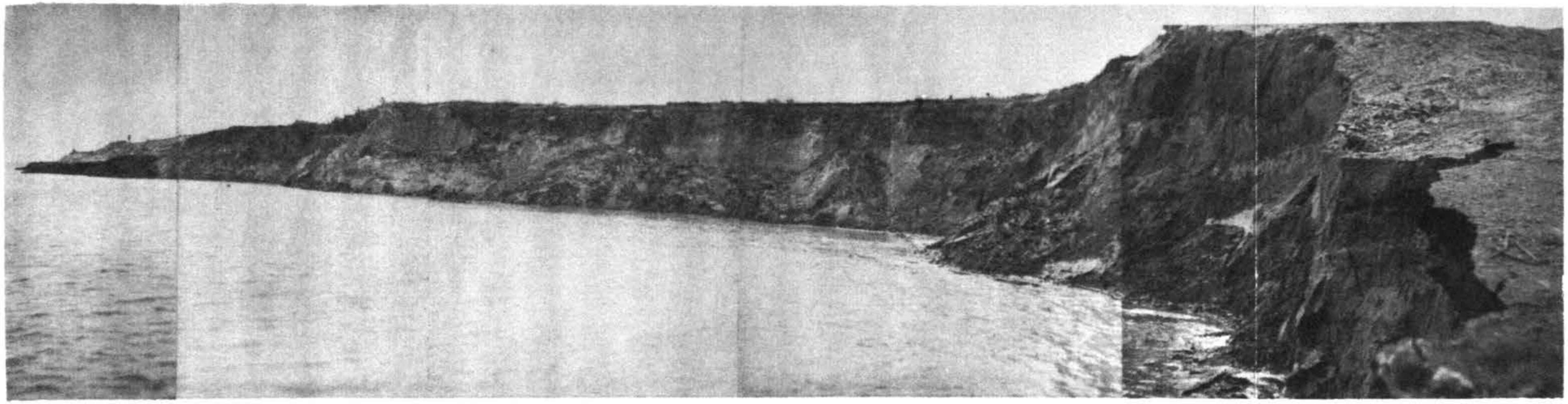
FAILURE AT RANGES 13-D—14-D



SOIL STRATA EXPOSED BY FAILURE AT RANGES 13-D — 14-D

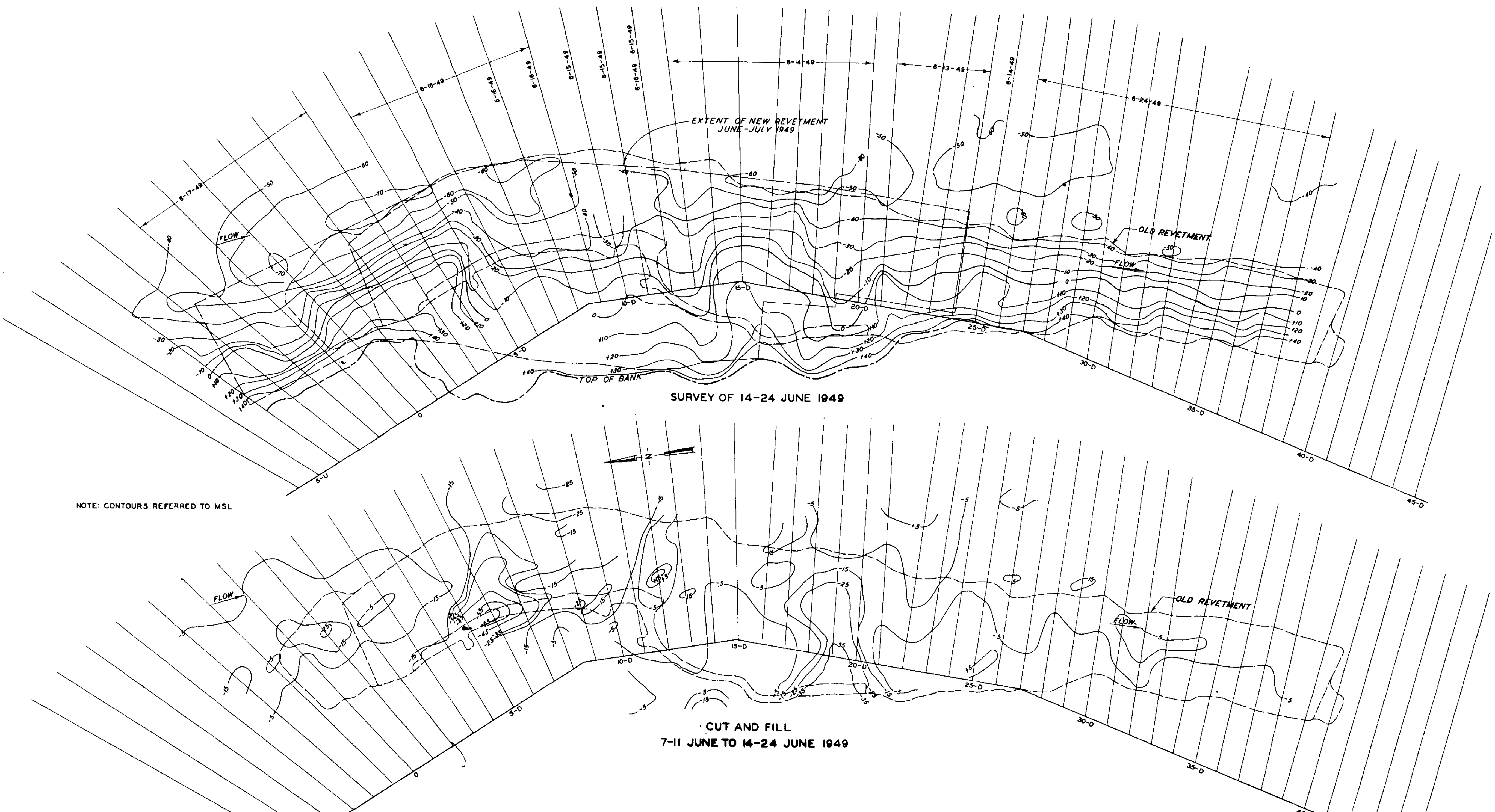
TAKEN FROM RANGE 12-D LOOKING DOWNSTREAM

14 JUNE 1949



FAILURE OF 13 JUNE 1949 AT RANGES 18-D — 21-D

TAKEN FROM RANGE 18-D — 14 JUNE 1949



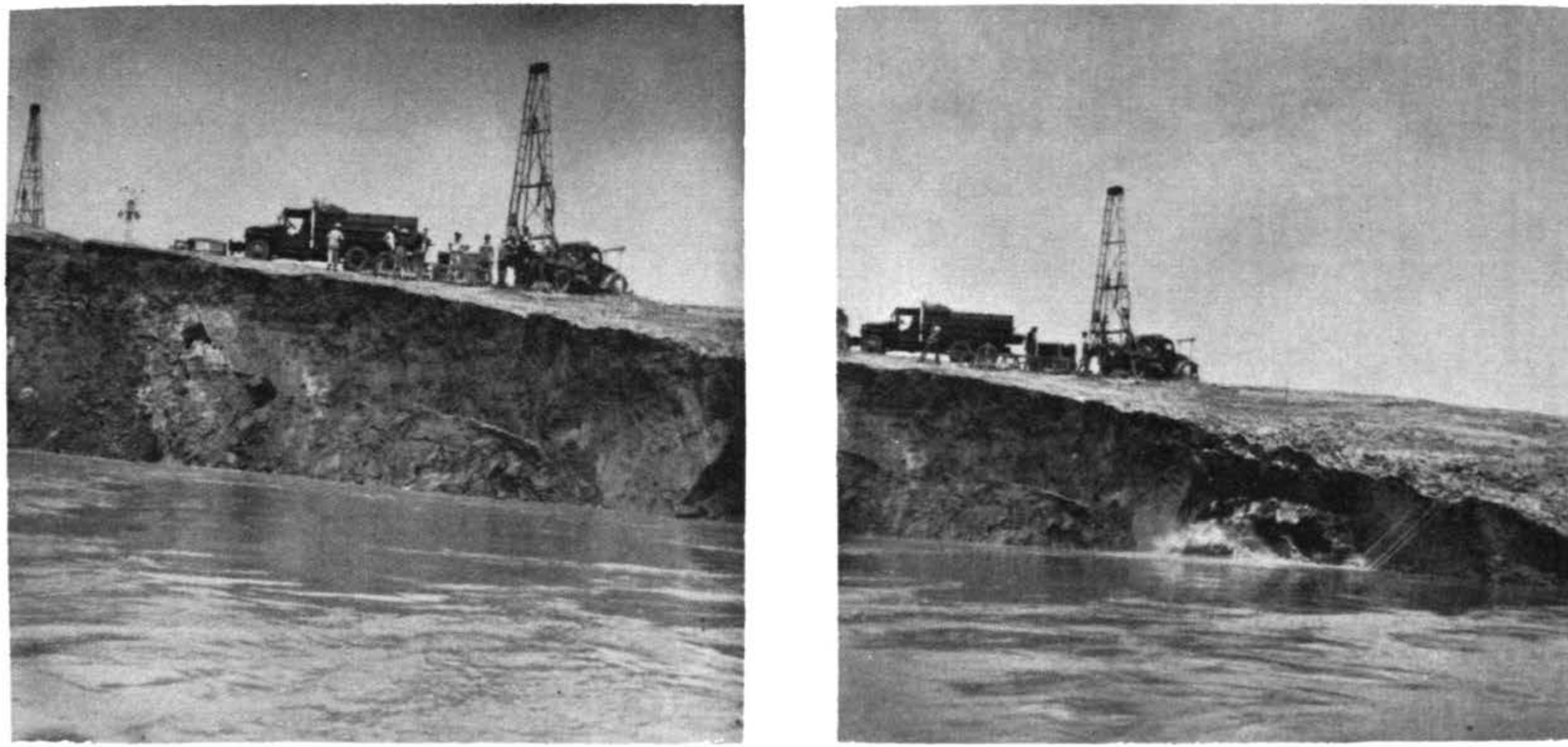
HYDROGRAPHIC SURVEY OF 14-24 JUNE 1949

CUT AND FILL 7-11 TO 14-24 JUNE 1949

SCALE

500 0 500 FT

PLATE 19



BANK CAVING NEAR BORING R-2-3

17 JUNE 1949

BEFORE

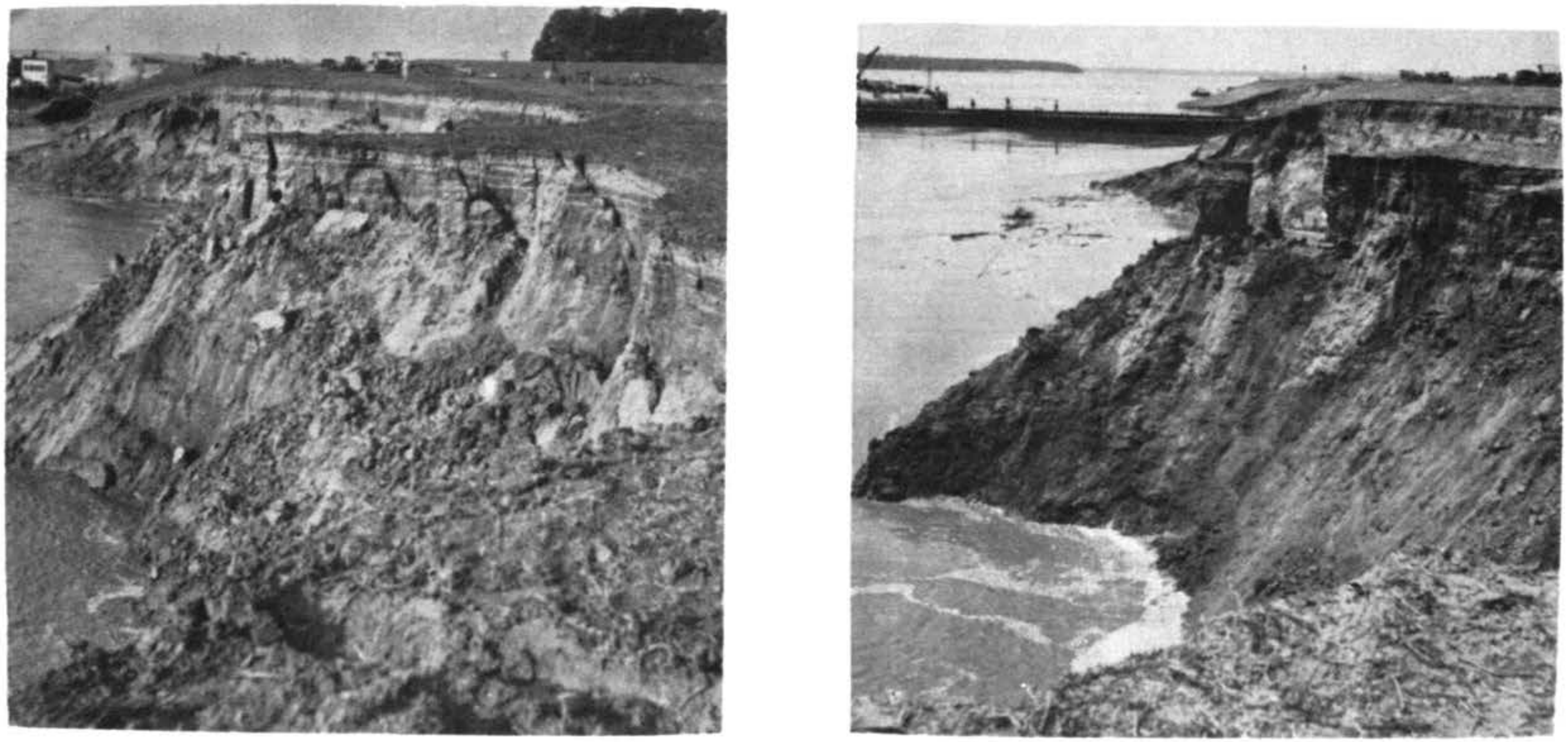
DURING

APPROXIMATELY 5 MINUTES ELAPSED TIME

AFTER

BANK CAVING AT RANGE 3-D

18 JUNE 1949

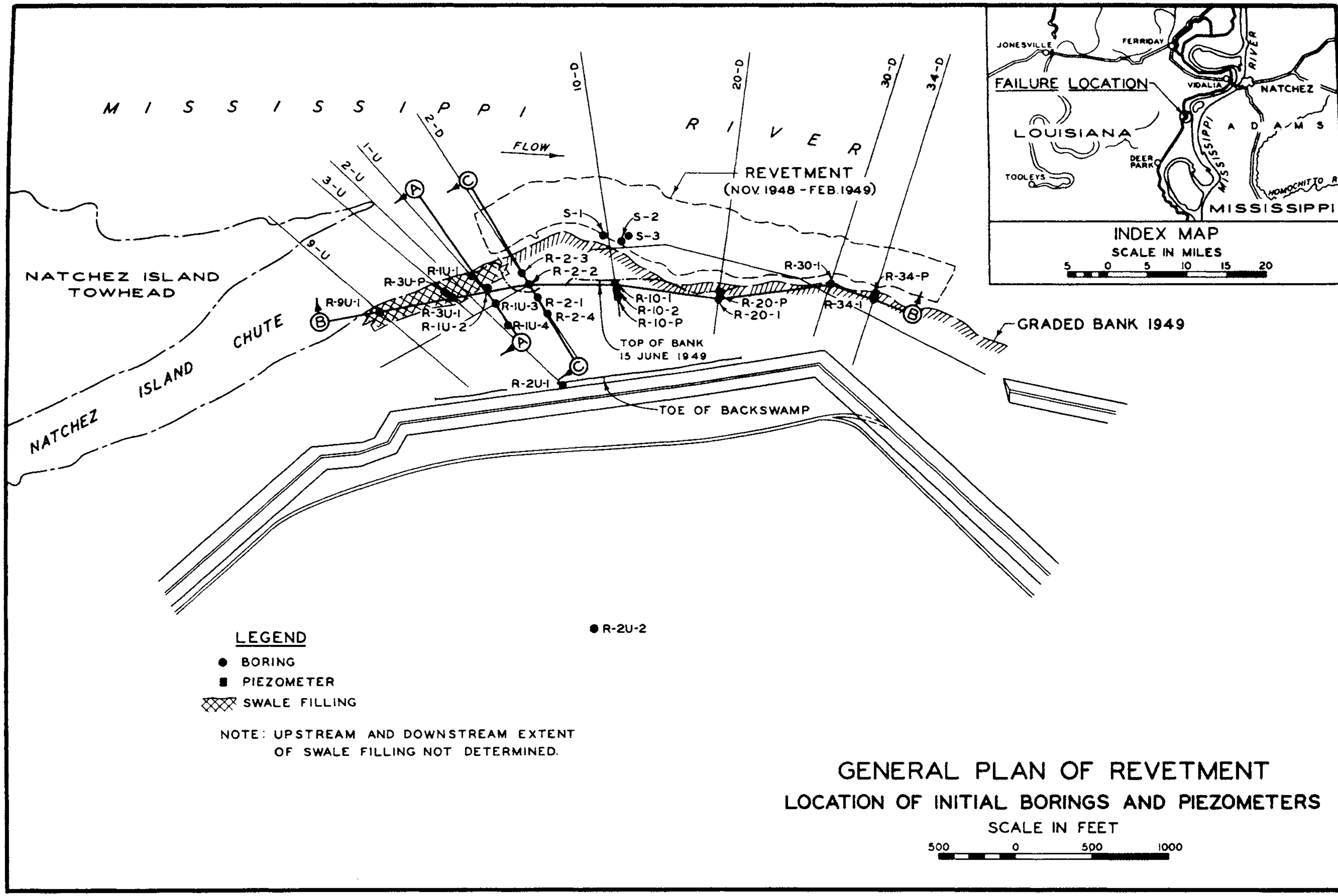


BANK CONDITIONS RANGE 2-D LOOKING DOWNSTREAM

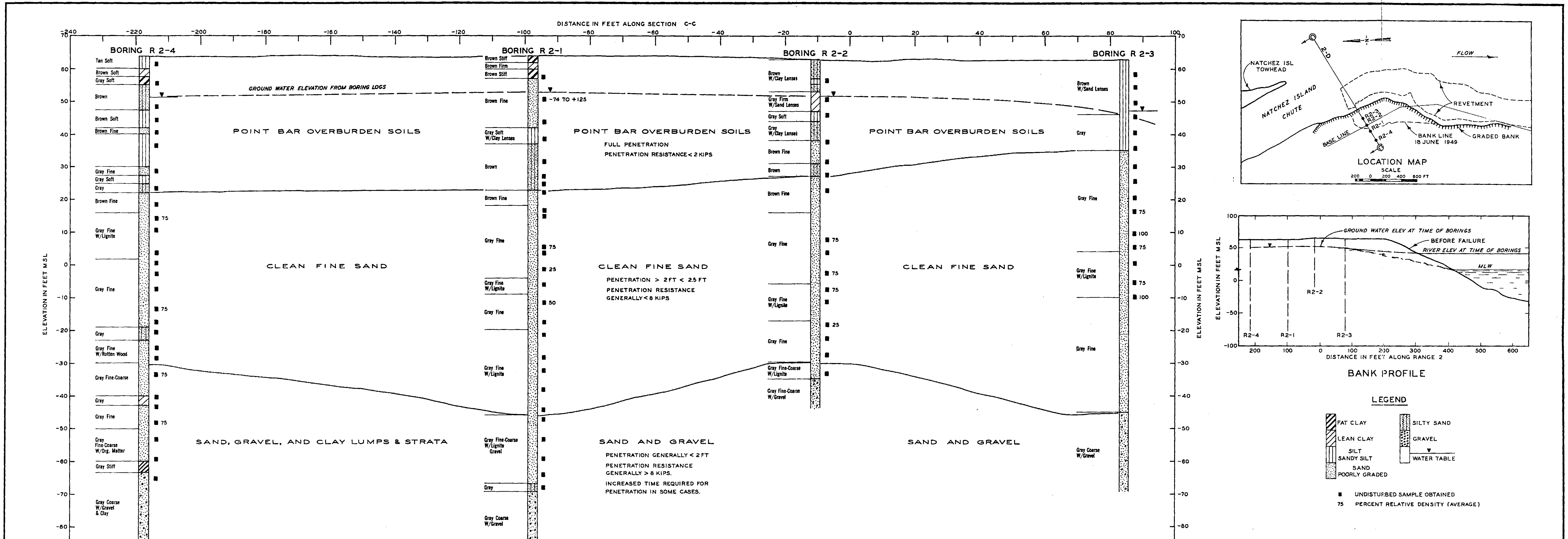
19 JUNE 1949

BANK CAVING AT RANGE 3-D

19 JUNE 1949

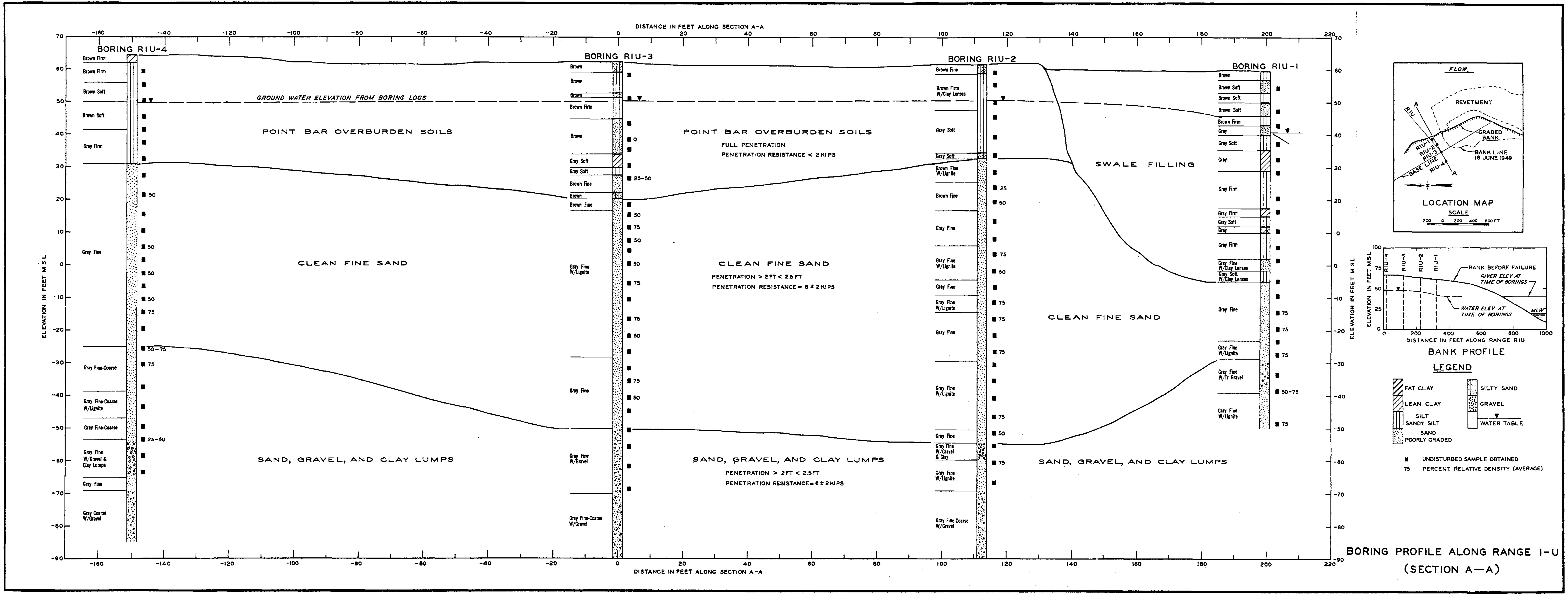


GENERAL PLAN OF REVETMENT
LOCATION OF INITIAL BORINGS AND PIEZOMETERS

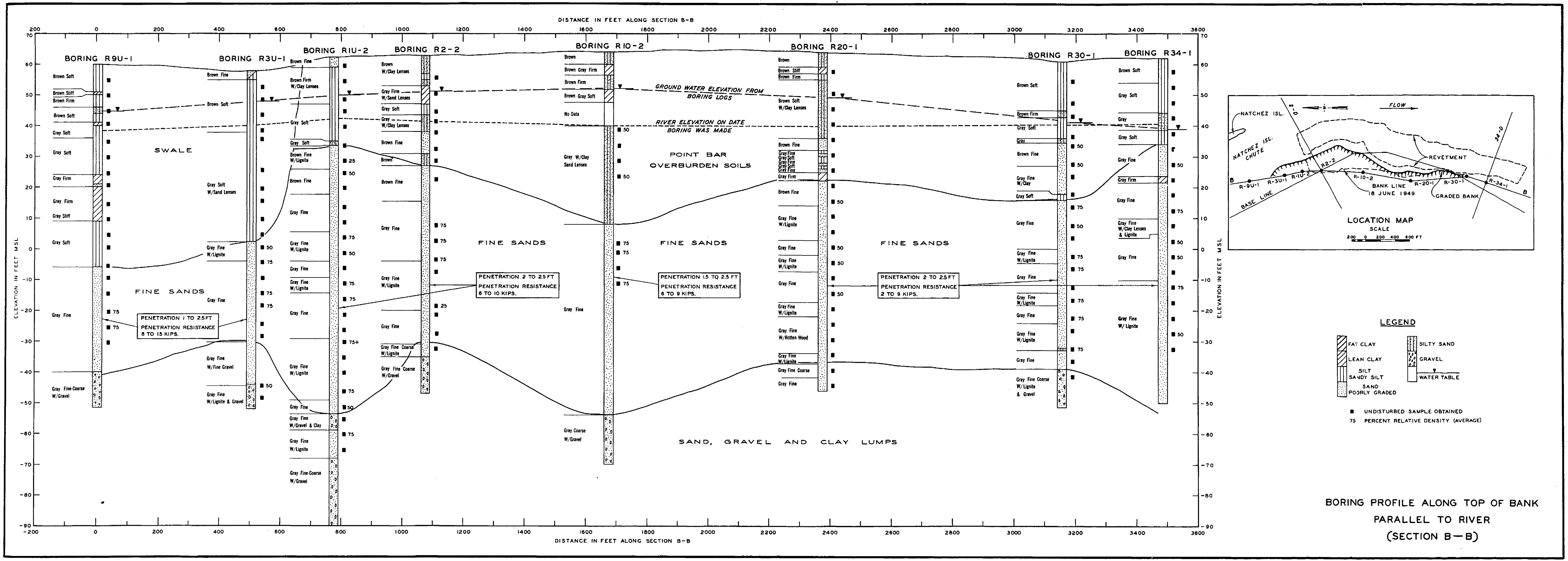


BORING PROFILE ALONG RANGE 2-D
(SECTION C-C)

PLATE 24

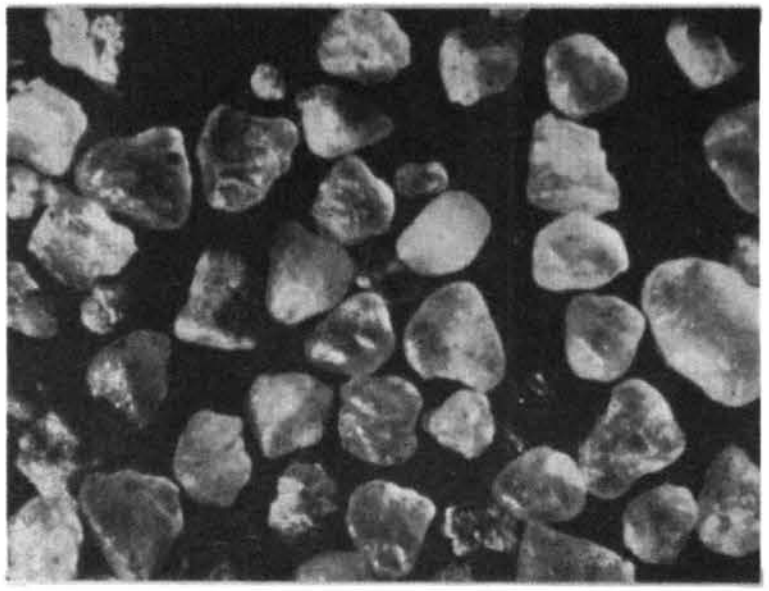


BORING PROFILE ALONG RANGE I-U
(SECTION A—A)

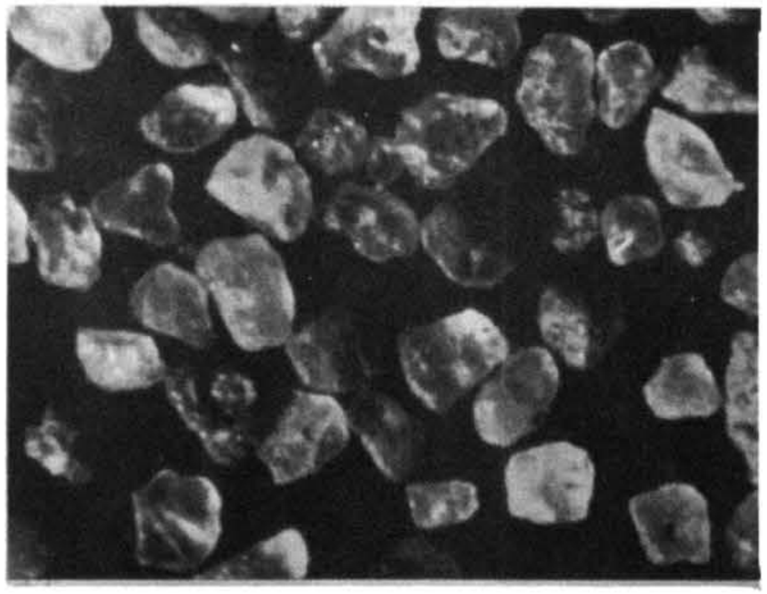


BORING PROFILE ALONG TOP OF BANK PARALLEL TO RIVER (SECTION B—B)

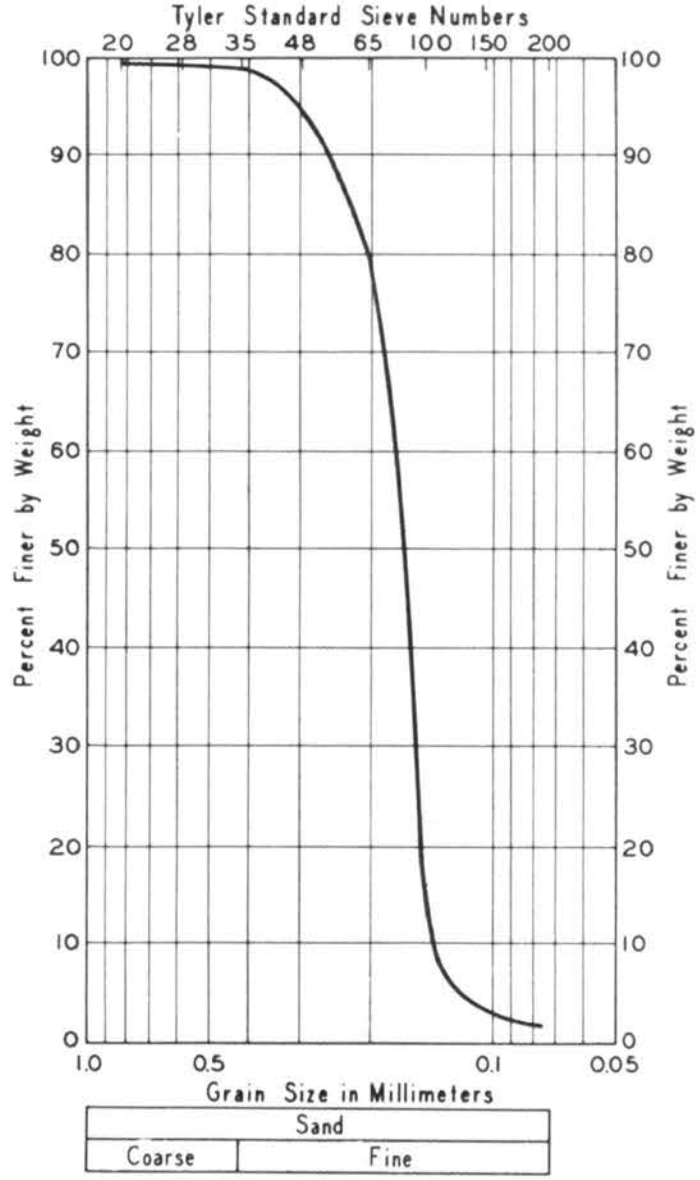
PLATE 26

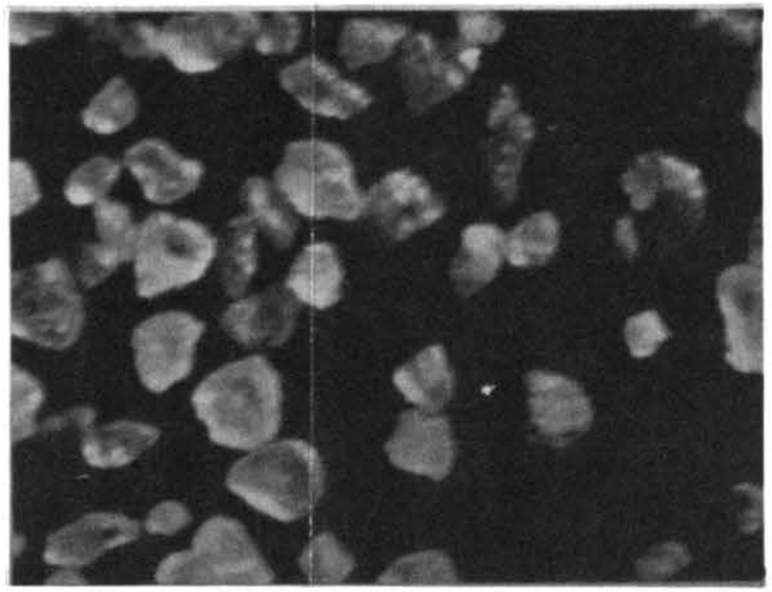


SHAPES BETWEEN TYLER SIEVE
SIZES 14 – 35



SHAPES BETWEEN TYLER SIEVE
SIZES 35 – 65





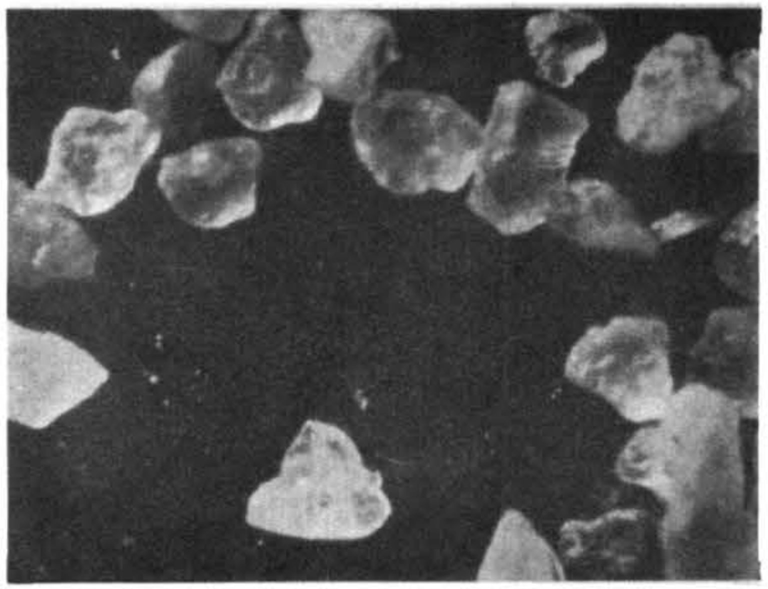
SHAPES BETWEEN TYLER SIEVE
SIZES 65 – 150

NOTE: MAGNIFICATION VARIED TO OBTAIN SAME GRAIN SIZE.

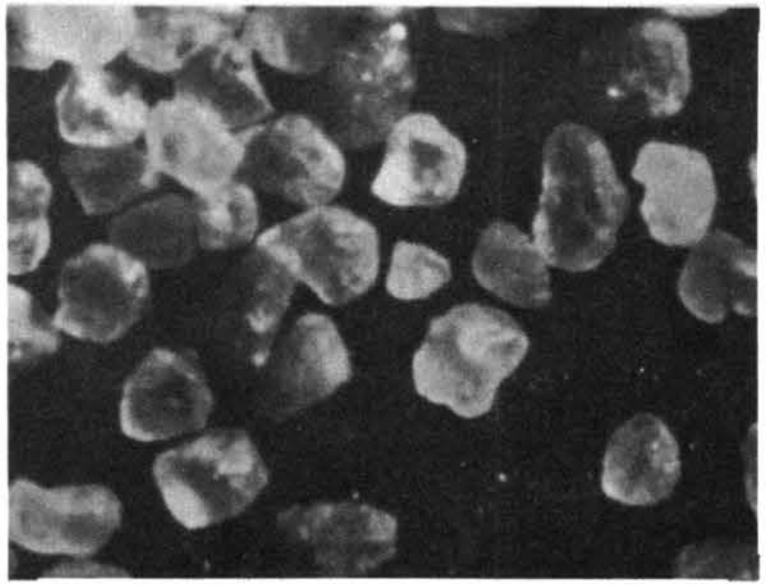
EQUIVALENT GRAIN SIZE DISTRIBUTION
AND GRAIN SHAPES

BORING R2 – 3 SAMPLE 14

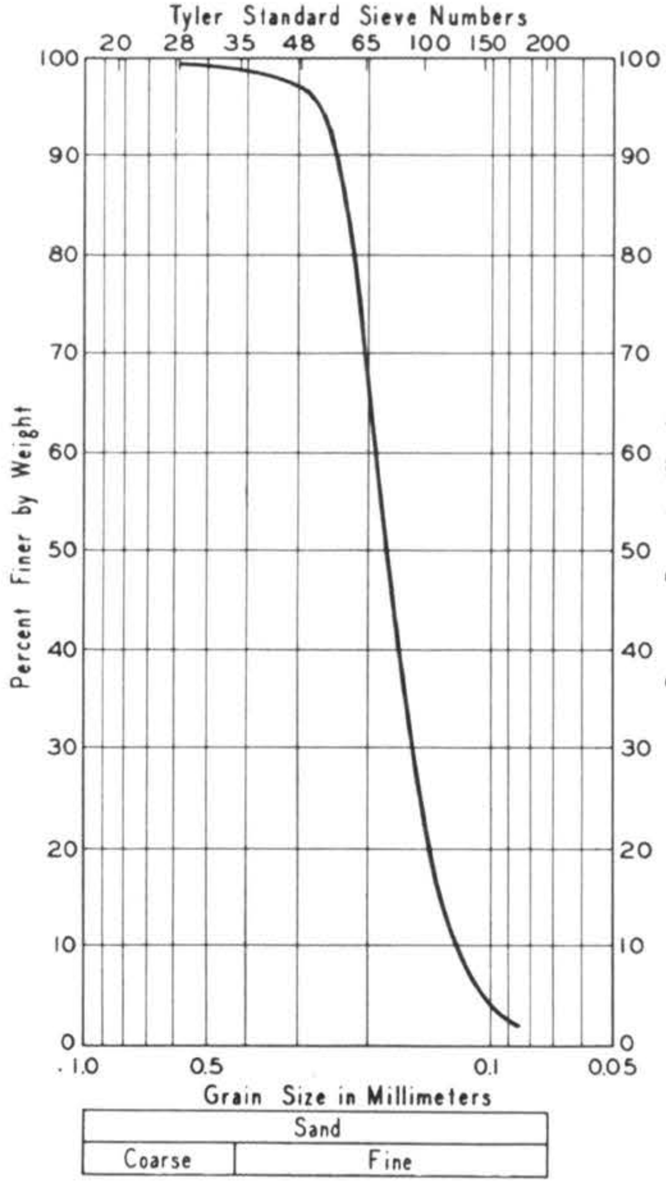
ELEVATION +5 FT MSL

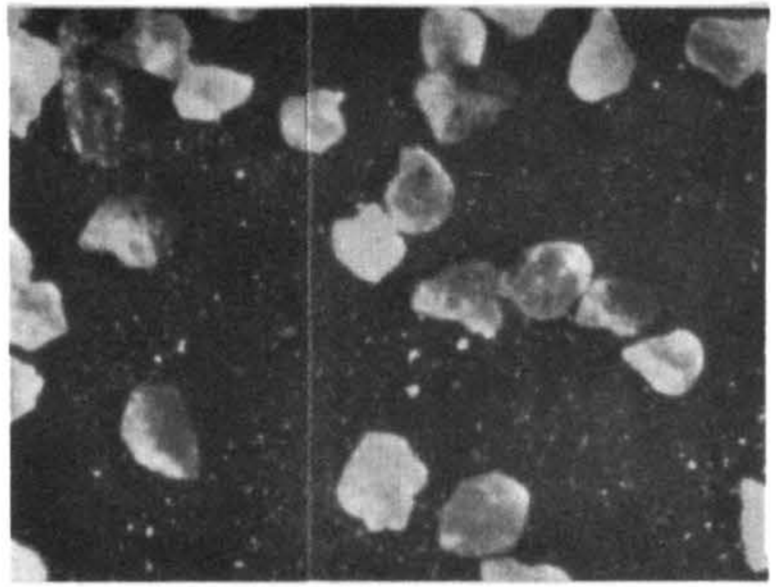


SHAPES BETWEEN TYLER SIEVE
SIZES 20 – 48



SHAPES BETWEEN TYLER SIEVE
SIZES 48 – 65





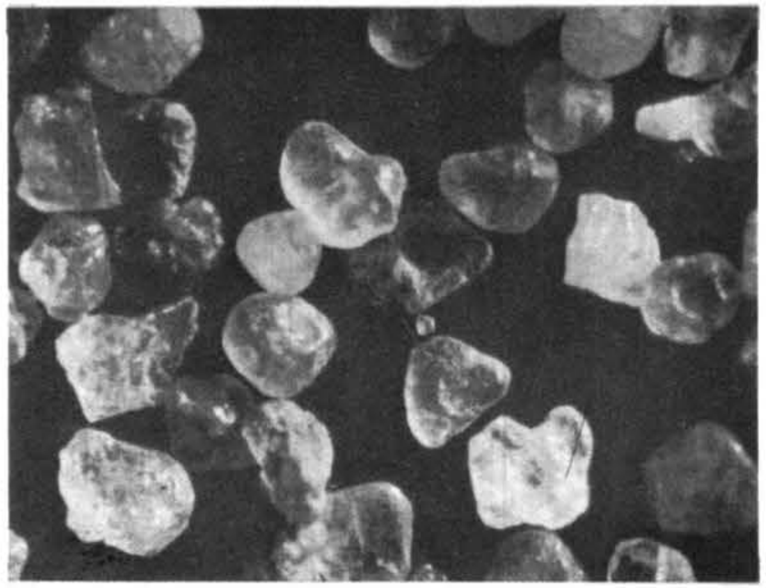
SHAPES BETWEEN TYLER SIEVE
SIZES 65–100

NOTE: MAGNIFICATION VARIED TO OBTAIN SAME GRAIN SIZE.

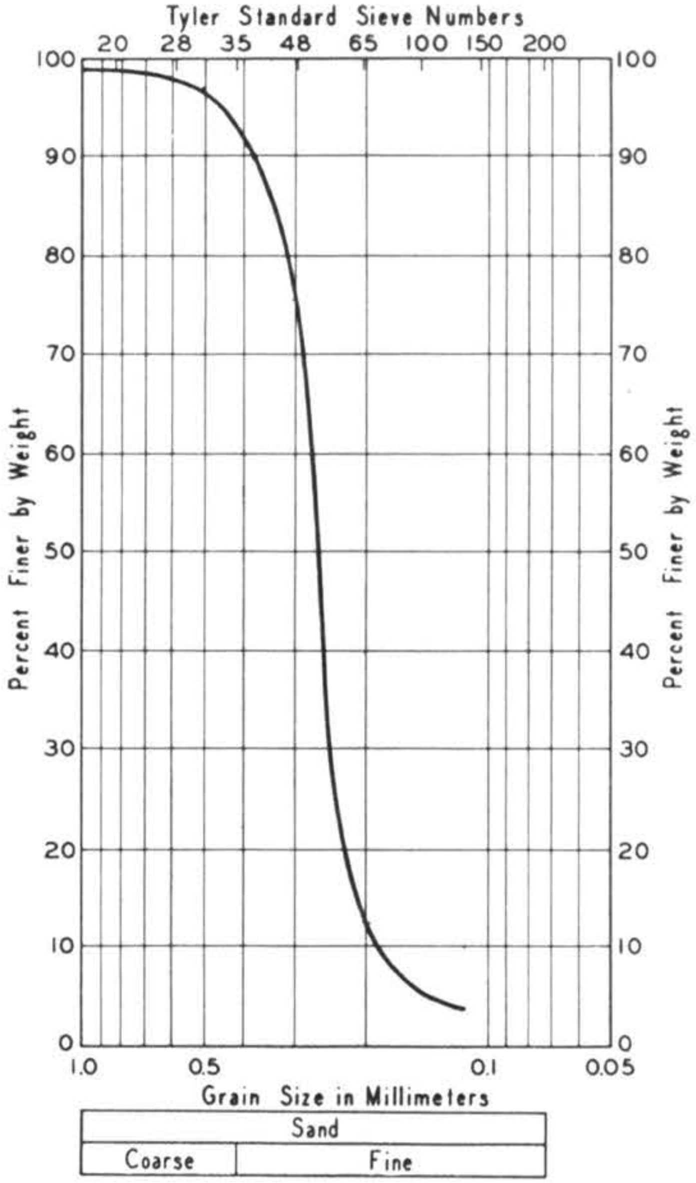
EQUIVALENT GRAIN SIZE DISTRIBUTION AND GRAIN SHAPES

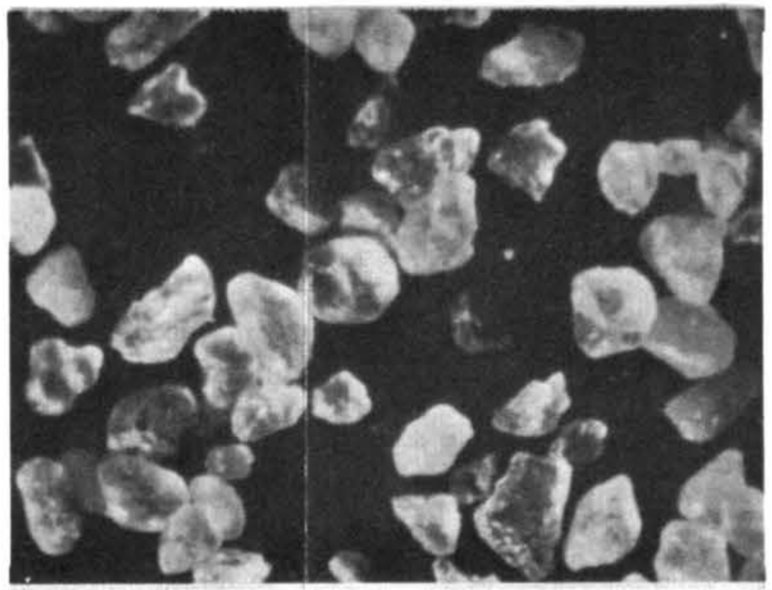
BORING R2–2 SAMPLE 15

ELEVATION +3FT MSL



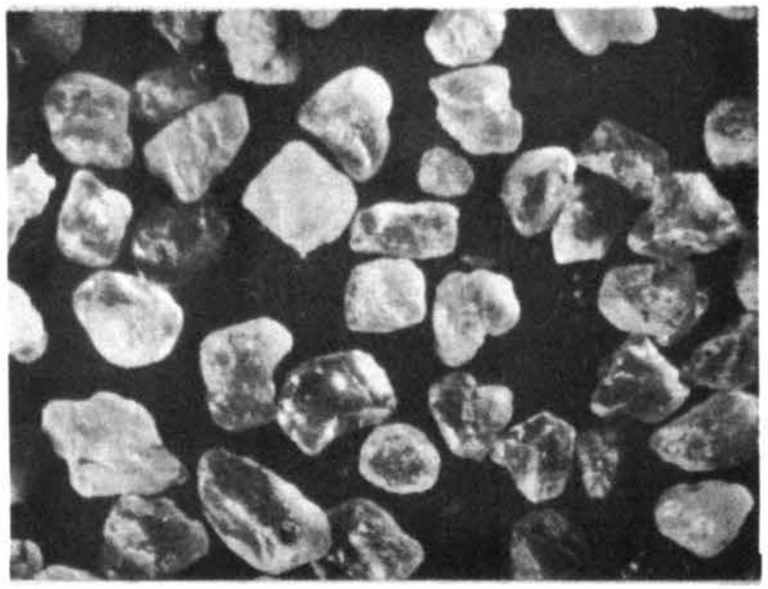
SHAPES BETWEEN TYLER SIEVE
SIZES 10 – 35





SHAPES BETWEEN TYLER SIEVE
SIZES 65 – 150

NOTE: MAGNIFICATION VARIED TO OBTAIN SAME GRAIN SIZE.

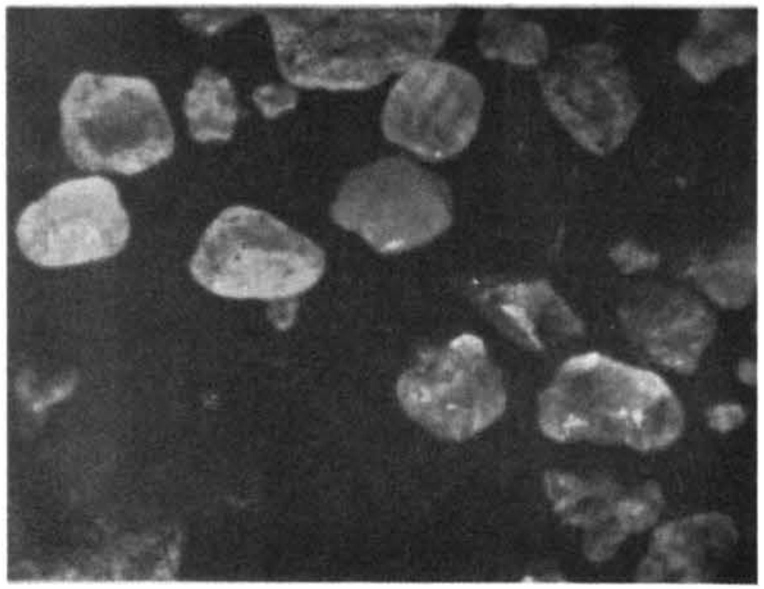


SHAPES BETWEEN TYLER SIEVE
SIZES 35 – 65

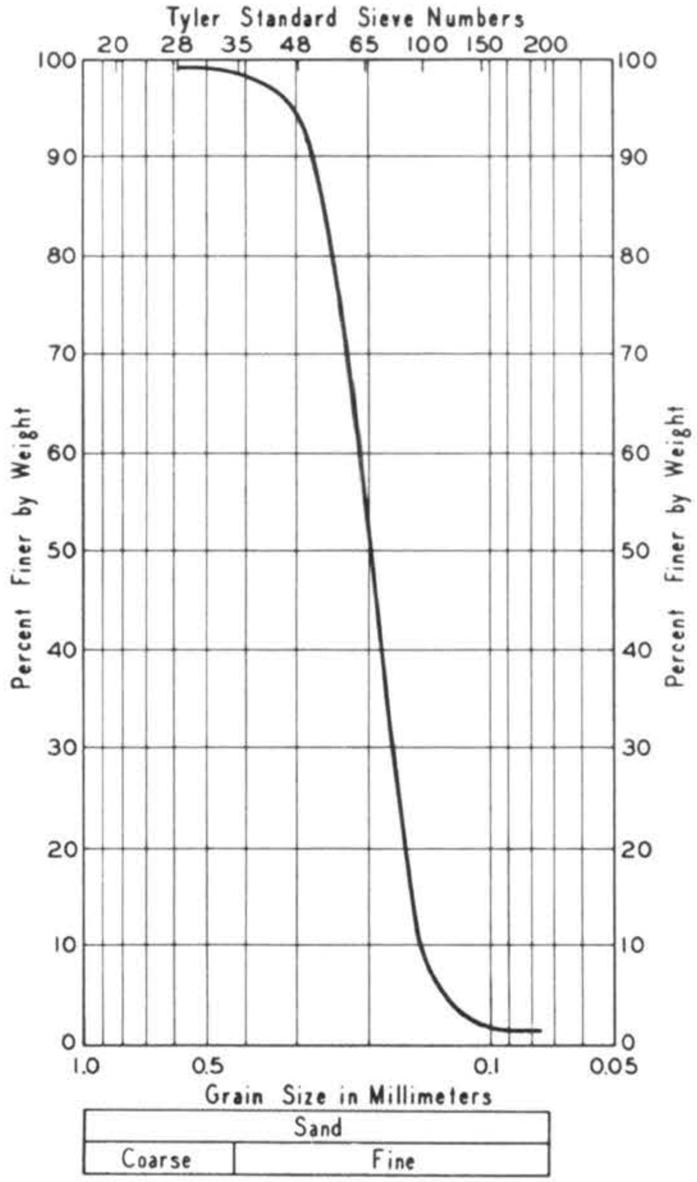
EQUIVALENT GRAIN SIZE DISTRIBUTION
AND GRAIN SHAPES

BORING R2 – 1 SAMPLE 19

ELEVATION +5 FT MSL

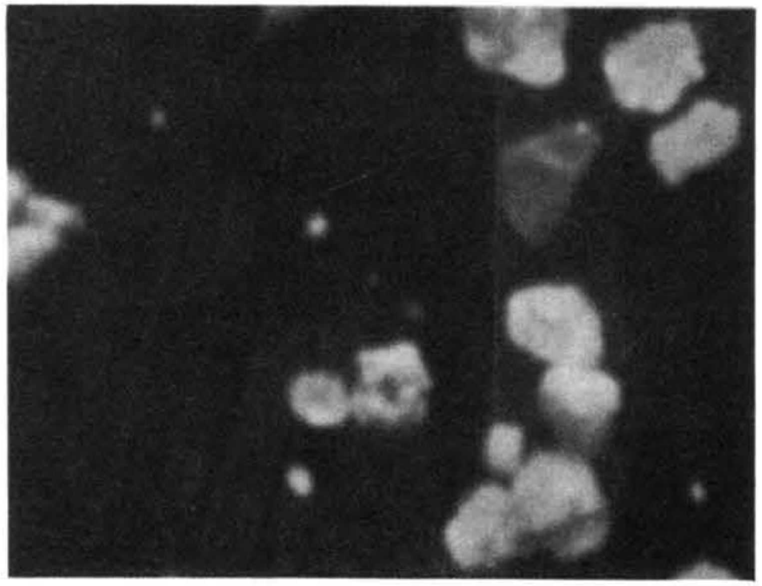


SHAPES BETWEEN TYLER SIEVE SIZES 20 - 35



SHAPES BETWEEN TYLER SIEVE SIZES 65 - 150

NOTE: MAGNIFICATION VARIED TO OBTAIN SAME GRAIN SIZE.



SHAPES BETWEEN TYLER SIEVE SIZES 35 - 65

EQUIVALENT GRAIN SIZE DISTRIBUTION AND GRAIN SHAPES

BORING R2-4 SAMPLE 24

ELEVATION +1 FT MSL

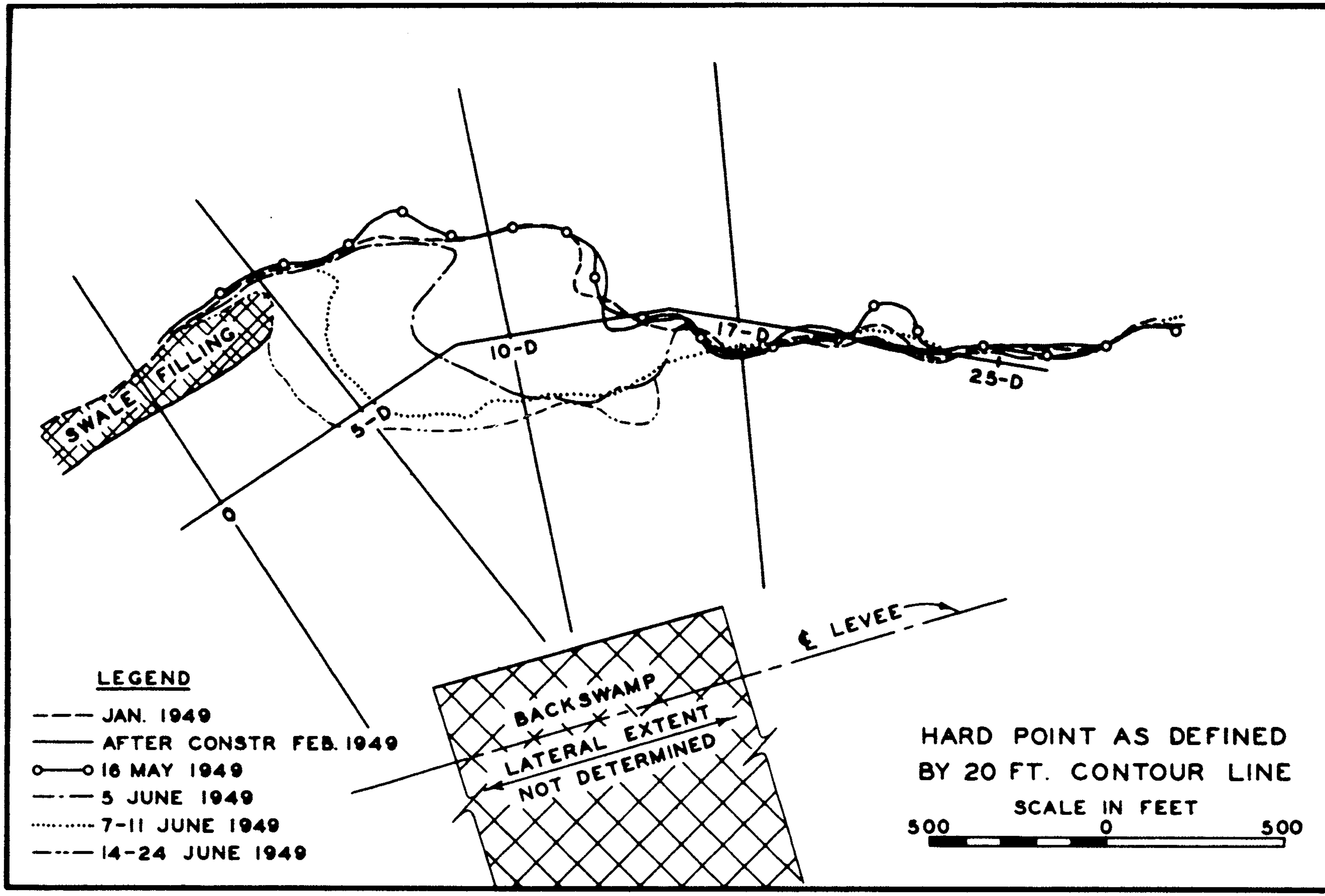
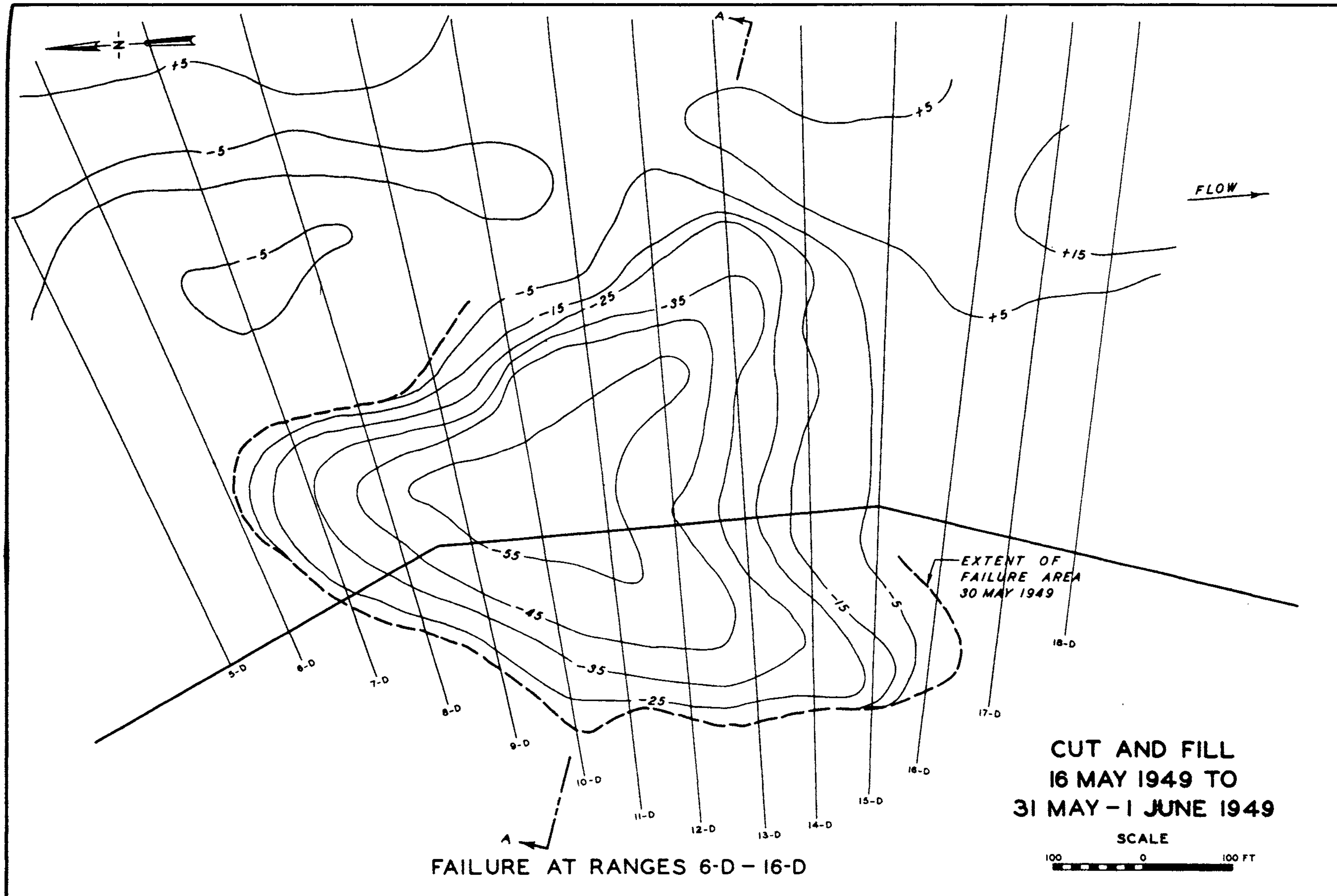

SWALE FILLING
10-D
17-D
25-D
5-D
0
℄ LEVEE
BACKSWAMP
LATERAL EXTENT
NOT DETERMINED
LEGEND
JAN. 1949
AFTER CONSTR FEB. 1949
16 MAY 1949
5 JUNE 1949
7-11 JUNE 1949
14-24 JUNE 1949
HARD POINT AS DEFINED
BY 20 FT. CONTOUR LINE
SCALE IN FEET
500
0
500

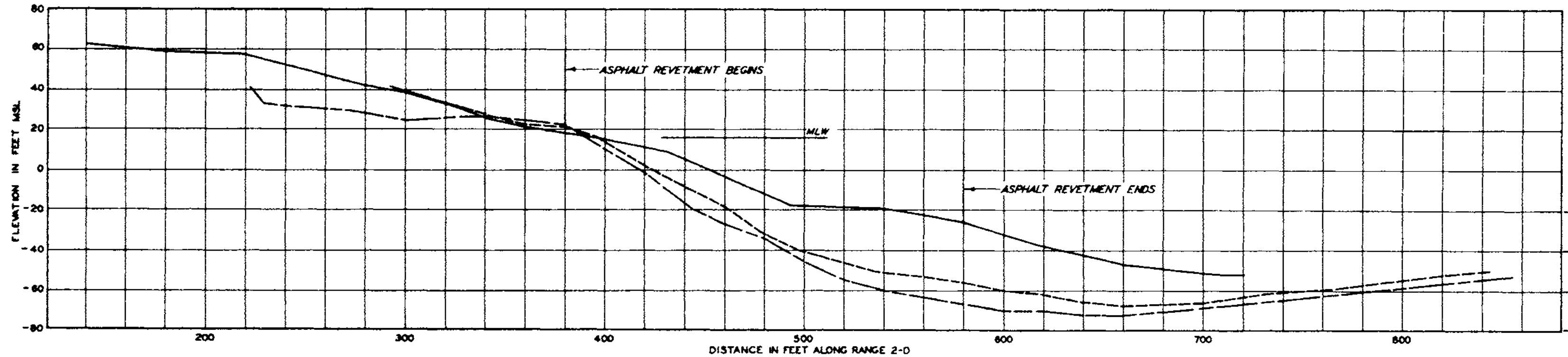
PLATE 31

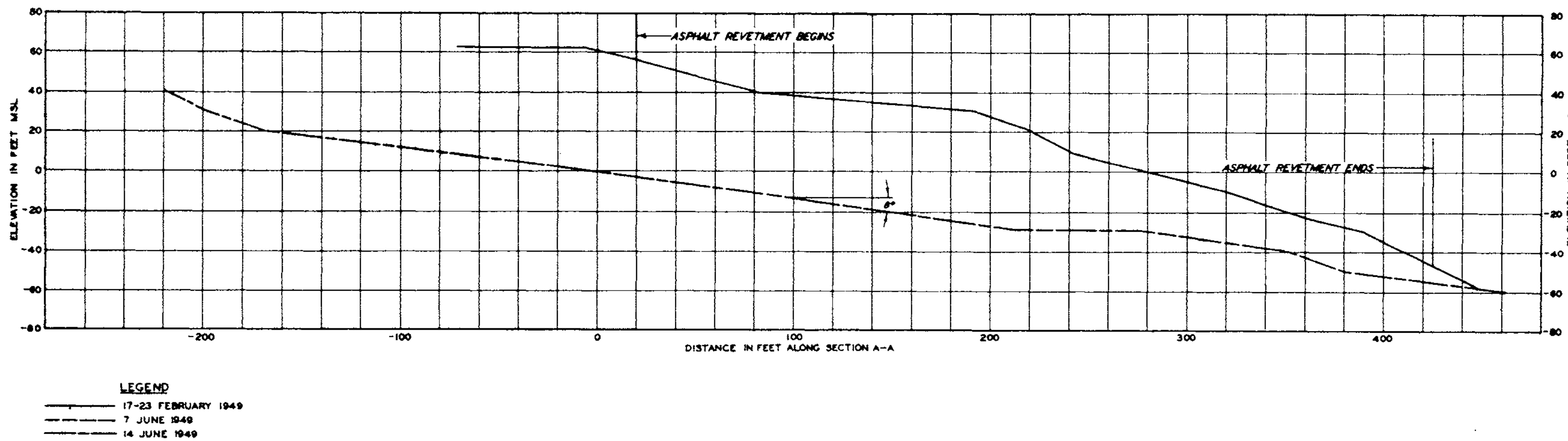


FAILURE AT RANGES 6-D – 16-D

CUT AND FILL
16 MAY 1949 TO
31 MAY – 1 JUNE 1949

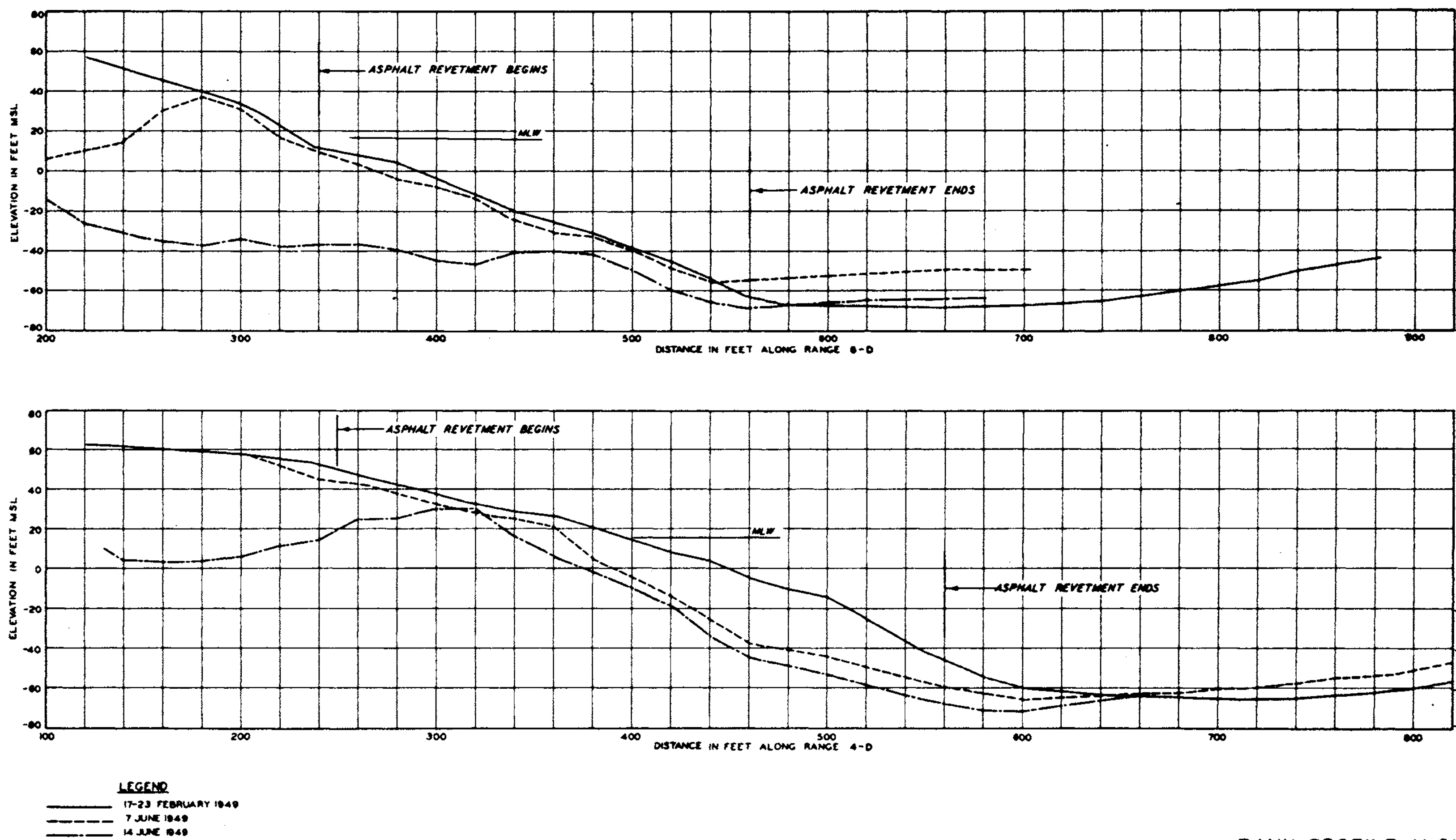
PLATE 32





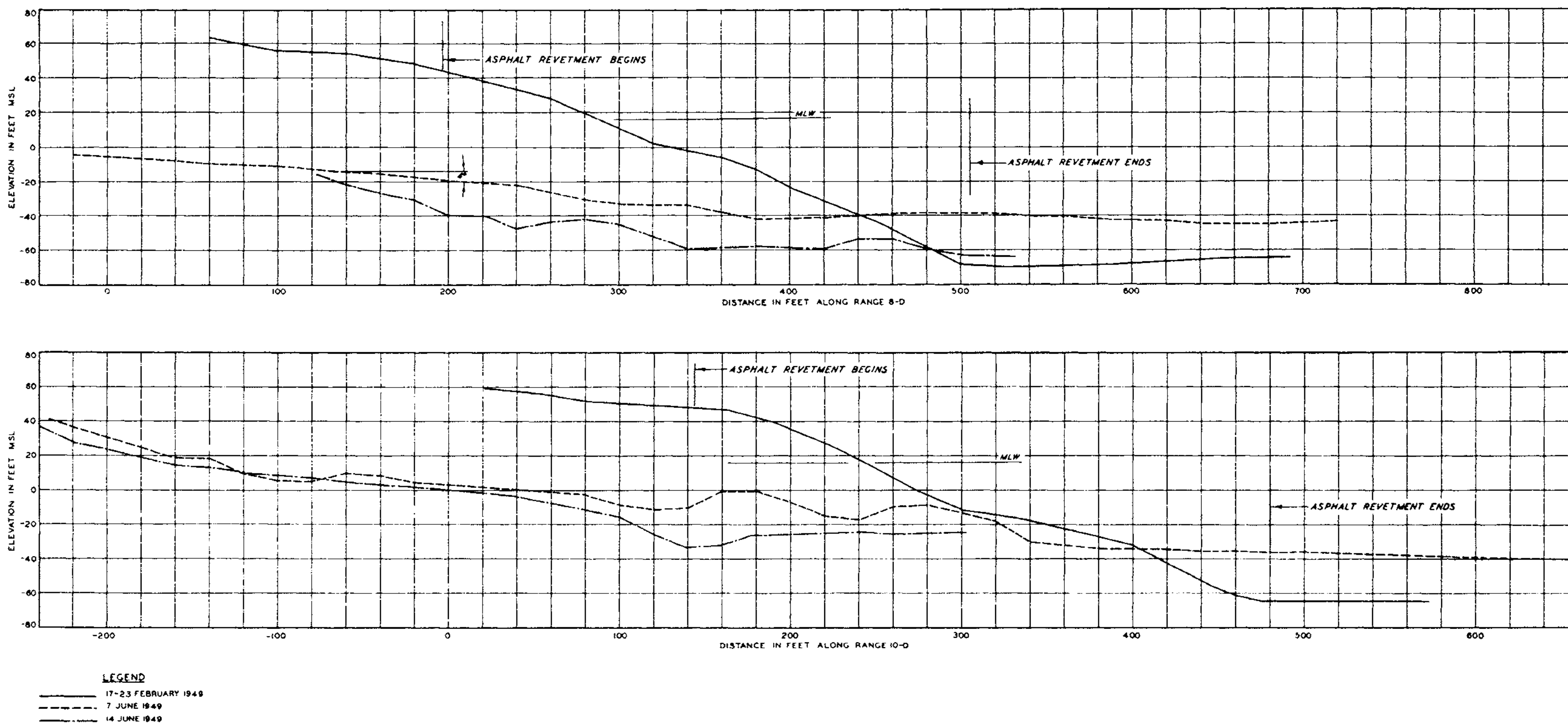
BANK PROFILES ALONG
RANGE 2-D AND SECTION A-A

PLATE 33



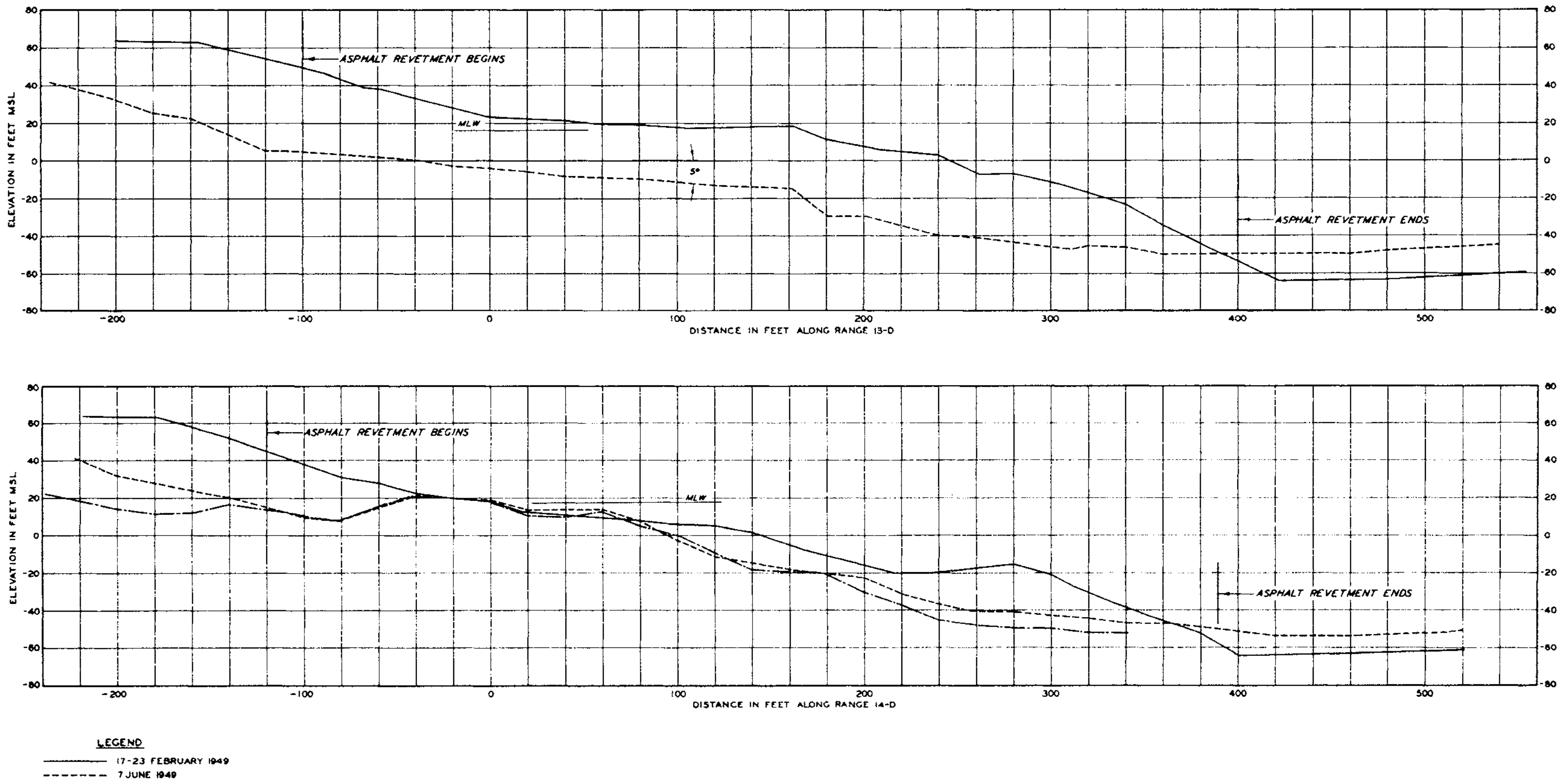
BANK PROFILE ALONG
RANGES 4-D AND 6-D

PLATE 34



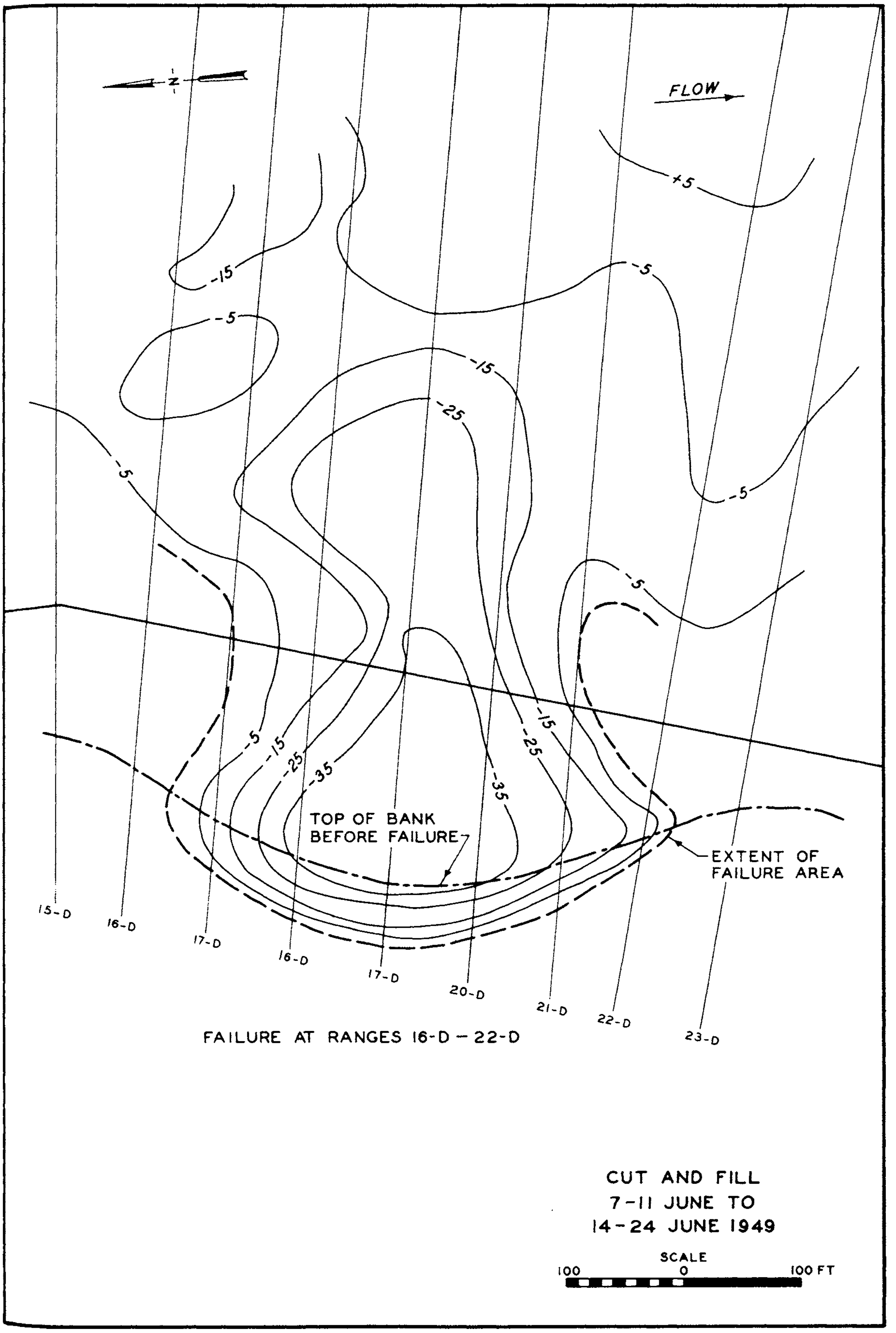
BANK PROFILE ALONG
RANGES 8-D AND 10-D

PLATE 35



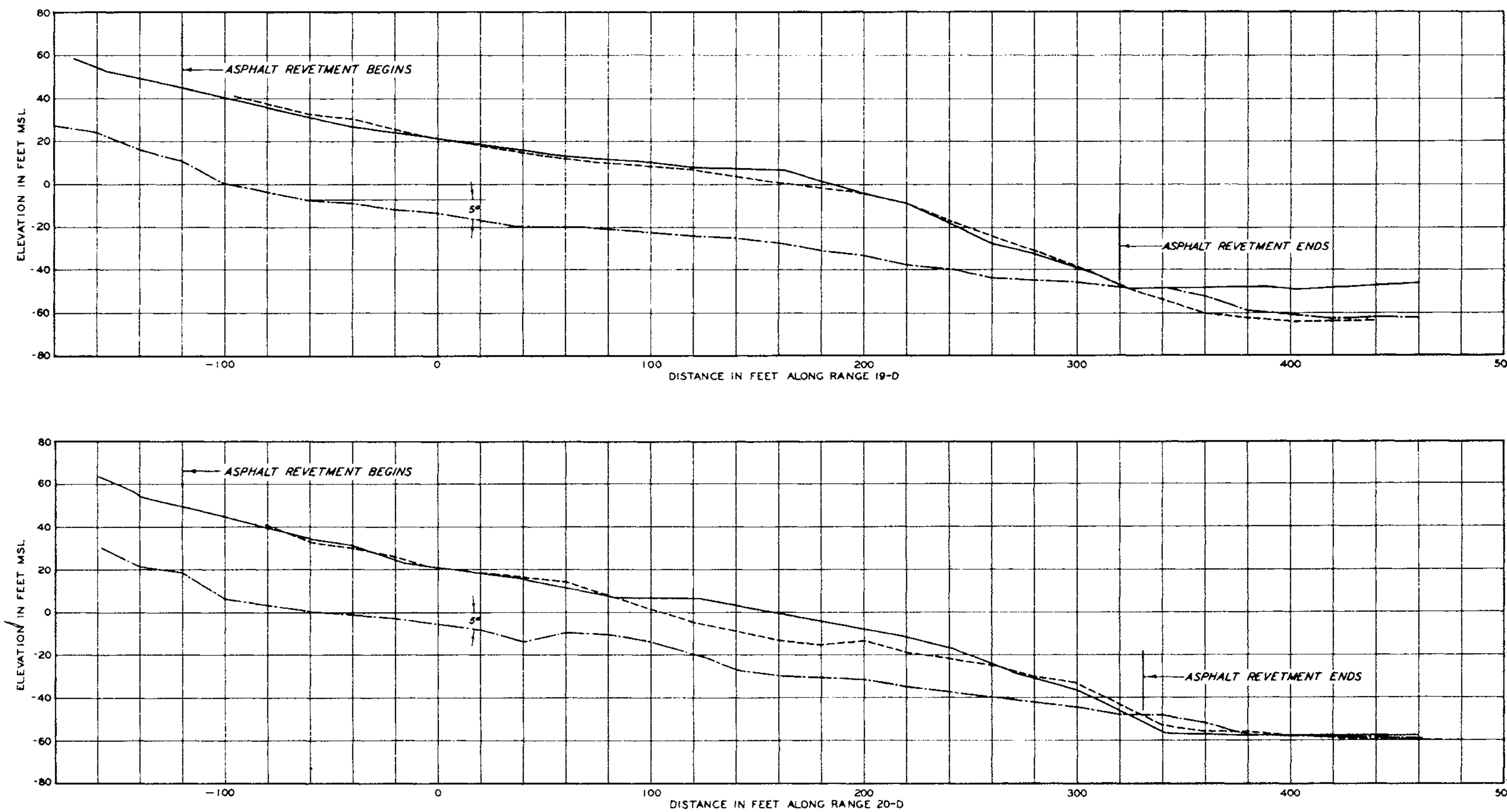
BANK PROFILES ALONG
RANGES 13-D AND 14-D

PLATE 36



FAILURE AT RANGES 16-D – 22-D

CUT AND FILL
7-11 JUNE TO
14-24 JUNE 1949



LEGEND

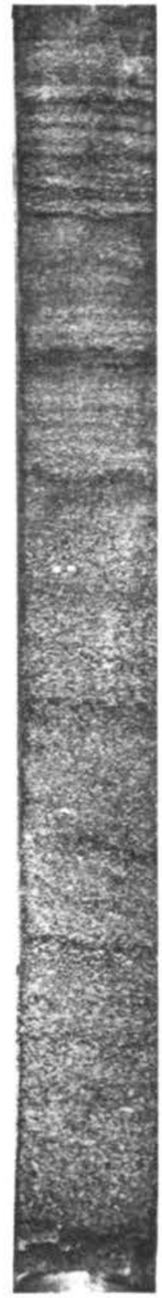
——— 17-23 FEBRUARY 1949
– – – – 7 JUNE 1949
—·—·— 14 JUNE 1949

BANK PROFILES ALONG
RANGES 19-D AND 20-D

PLATE 38

TYPICAL SAMPLES

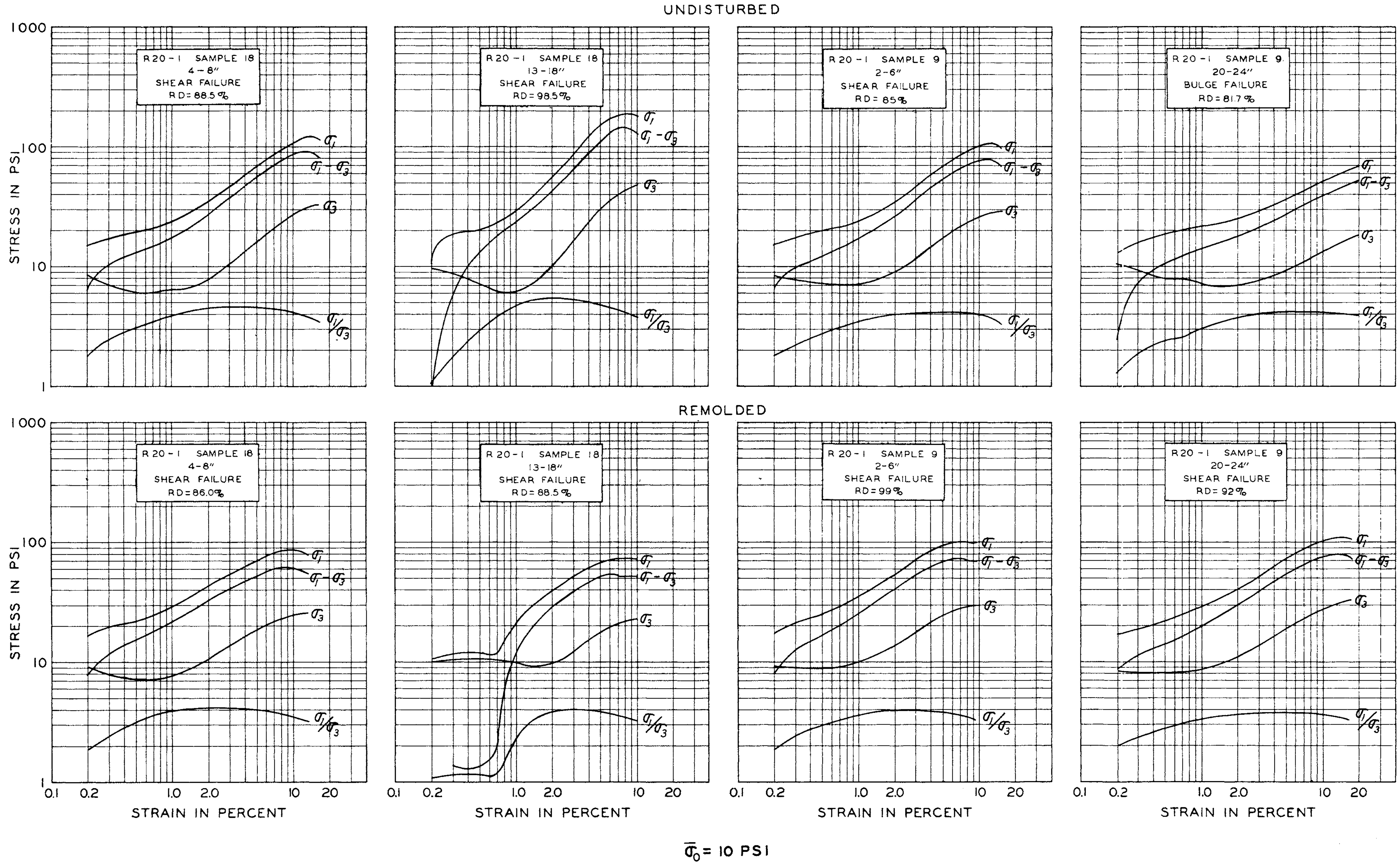
STRATIFIED



PHOTOGRAPHS OF SAMPLES

R 30 - 1 SAMPLE 18

R 3U - 1 SAMPLE 17



TRIAXIAL TEST ON SANDS
STRESS STRAIN RELATIONS
CONSTANT VOLUME TEST

APPENDIX: DETAILS OF FIELD AND LABORATORY INVESTIGATION

PART I: DISCUSSION OF SAMPLES

New Sampling Procedure*

1. In sampling operations during this investigation it was found that the use of a viscous, commercial-type drilling mud in the bore hole supported the sides of the boring and eliminated the necessity of using casing. It was also found that when borings were filled with drilling mud saturated sand samples remained in the sampling tube and could be easily recovered. Recovery of undisturbed sand samples had not been possible in the past as plain water or natural muds of low viscosity had been used in boring operations. No unusual procedures were required in the sampling operations, but since the sample was contained in a seamless steel tube new techniques were required for handling the sample and for its removal from the tube. Considerably more care is required in the case of sands, as compared to cohesive soils, as a slight shock may easily cause appreciable damage to the sample.

Description of Samples

2. Most of the sand samples were highly stratified, as shown on plate A1 which illustrates typical samples, and in most cases no disturbance could be detected that could be attributed to sampling operations. In a few samples that had been shipped early in the investigation

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* Described in detail in Waterways Experiment Station Bulletin No. 35 "Undisturbed Sand Sampling Below the Water Table."

in a horizontal position, slumping had occurred normal to the axis of the tube in strata of what were evidently loose materials. Sample 15 of boring R2-3, as shown in plate A1, is typical of this type of disturbance due to shipping. This disturbance generally occurred in alternate strata and could be detected only when the slice was made perpendicular to the plane that had been horizontal during shipment of the sample. Later samples were shipped in a vertical position and were packed more carefully.

3. Two Shelby tube samples were obtained which were unusual in the following respects. Sample 14 of boring R1U-1 contained shear planes, as shown on plate A2. The arrangement of the soil strata on either side of the planes is such as to indicate that shear planes are probably due to natural causes rather than to sampling operations. This sample came from the lower part of the swale filling on range 1-U. Sample 17 of boring R2-4 contained an open channel extending down through the top half of the sample. This sample is also shown on plate A2. The upper part of the channel had been filled with paraffin which had run down into the sample when the upper end was sealed. The remainder of the channel was open. It had a very smooth, firm surface and gave the impression that it had been formed by flowing water. There are two possible explanations for this formation: One, water had percolated up through the sample so rapidly during sampling operations that an open channel was formed. This is borne out to some degree by the disturbance of the strata near the lower end of the channel. Two, the channel may have been formed by natural percolation of ground water some time prior to sampling; i.e., the channel may be evidence that internal erosion,

on an extremely small scale, was occurring in the clean sands prior to the time of failure. The detection of this definite void space may provide an explanation of some of the unusally low densities obtained in this investigation.

Effects of drilling mud on sample

4. Preliminary laboratory tests indicated that the drilling mud would not penetrate the sands an objectionable amount. The deep sands and the drilling mud were both of approximately the same color and, in a few cases, the penetration of the drilling mud was not obvious by inspection of the sliced samples. This was especially true in the non-stratified or only slightly stratified samples as shown on plate A3. This type of sample was not common; in most cases the penetration of the drilling mud could be detected by the darker color, caused by the retention of water in the drilling mud. Samples of this type are shown on plate A4. The drilling mud was dyed in boring R20-1 by adding gentian violet to the mixture to obtain positive information on the depth of penetration. The penetration of the drilling mud was very obvious in this case, as is shown on plate A4. Little or no penetration occurred at the bottom of the sample.

Stratification

5. As indicated above, the clean sands were highly stratified. Individual strata of homogeneous sands of greater thickness than 1 ft were rare. These strata were generally tilted, sometimes as much as 30° from the horizontal. Considering the manner in which a point-bar deposit is built, it is not unreasonable to expect that the strata would

occur as relatively thin layers, and that thick, homogeneous strata would be the exception rather than the rule. The direction of the dip of the strata at Morville was not completely determined, since the orientation of the samples in the ground is not known.

6. Lignite was common, usually occurring in a finely divided form in very thin strata. Occasionally strata of pure soft lignite were encountered, however, which were several inches thick, and in other cases the lignite was distributed uniformly throughout the entire sample.

Penetration resistance

7. The penetration resistance of a cohesionless soil is quite often taken as a measure of its relative density. The resistance to penetration of the 3-in. Shelby tube and the time required for penetration were observed carefully and recorded. These data have been plotted as penetration indices on the detailed boring logs, plates A5-A21. Since the penetration resistances of the deep sands were in many cases equal to or greater than the capacity of the drill rigs, the total penetration resistance alone does not reflect changes in relative densities at these depths. However, the length of drive obtained with the maximum rig capacity varied and this may be considered the same measure of the comparative resistance. In some cases the penetration appears to be related to relative density, in that it was not possible to make full drives in the denser sands. However, penetration also appears to be related to several other factors. The penetration resistance appears to vary with soil type — in any given boring the resistance increases suddenly at the boundary between the overburden and the clean sands. The

total penetration resistance also seems to increase with depth in the clean sands (see boring RIU-3, plate A11). This may be due either to increasing confining pressures, to increasing grain size, to increasing relative densities, or to any combination of these factors. Exceptions to these relationships do exist in the data but it would appear reasonable to assume that penetration resistance varies with grain size, depth and relative density.

8. As stated previously, it was originally intended to make use of the penetration resistance as a measure of density in cases where it was not possible to obtain undisturbed samples, and it was expected that relatively few undisturbed samples could be taken. Due to the development of the new method of sampling it was possible to obtain an adequate number of undisturbed samples from which densities could be determined directly. Thus, the need for penetration resistance was eliminated insofar as density determinations were concerned, for this particular investigation. However, the penetration resistance data are of value in the development of sounding devices such as cone penetrometers and may be of assistance in interpreting the results of standard penetration tests (split-spoon sampler). The penetration resistances are shown on the logs of borings and are used in a qualitative sense in this report. A quantitative analysis of the penetration resistance of sands appears rather complex and is not considered warranted in this report as densities were also determined by direct methods. It is intended, however, to make a further study of these data in connection with other development work.

PART II: RESULTS OF FIELD INVESTIGATIONS

River Borings

9. Borings S1, S2, and S3 were made from a barge anchored in the failure area to determine if the asphalt mat had been carried away by the failure or if the soil had moved out from under the mat, allowing the mat to drop and yet still give protection to the bank. Borings S1 and S2 consisted of probing with a fishtail auger to locate any mat present. Casing was not used and no attempt was made to recover samples since no resistant material was encountered. Boring S3 was made in a casing. The water was 61 ft deep at the time the boring was made (river elev approximately 41 ft msl). Samples could not be obtained for the first 10 ft of the hole (-20 to -30 ft msl) but a fine grey sand was present in the wash water from this depth. At -30 to -31 ft msl a sample of fine grey silty sand was obtained, and at -31 to -32 ft msl a stratum of lean clay was encountered. The clay was badly fissured and had grass roots in it which appeared to have been growing in the soil only a short time previously. The same lean clay was encountered in scattered lumps to an elevation of -37 ft msl embedded in a fine grey sand. No mat was encountered, but the borings were limited to an area which was very near the top edge of the old mat. Additional borings could not be made at greater distances from the bank because of the presence of the bank grader in the area. No undisturbed soils were encountered in the 17-ft depth of boring S3 indicating that the material was probably fill from the failure.

Borings on Range R-2

Boring R2-3

10. Boring R2-3 was located 85 ft on the riverside of the base line on range 2-D. The detailed log of this boring is shown on plate A5. The overburden material extended to a depth of 28 ft (elev 35 ft msl) and consisted of silts and sandy silts having a cohesion, C_u, in the range of 0.2 to 0.4 ton per sq ft. These soils are not representative of the fatter soils which existed on the relatively hard point riverward from this boring. Nine Shelby tube samples were taken in the clean sands between the depth of 28 and 74 ft. The relative densities as determined on 5 of 9 samples varied between 50 per cent and greater than 100 per cent except for three specimens which had negative values of relative density. In this case, the low values were due to high concentrations of lignite in those specimens. Sampling was stopped at 80 ft and the remainder of the hole was advanced with a fishtail auger. This was necessitated by bank caving and the operation of the bank paving plant in the area. The area where this boring was made caved into the river about 18 June.

Boring R2-2

11. Boring R2-2 was located on the base line on range 2-D. A detailed log of this boring is shown on plate A6. The overburden soils extended to a depth of 36 ft (elev +27 ft msl) and consisted of fine sandy soils with the exception of a stratum of lean clay at 10 to 15 ft depth. These soils had low cohesion, C_u, in the order of 0.2 to 0.3 ton per sq ft.

12. Twelve drives were made in the clean sands from which undisturbed samples were recovered except for three samples which were lost. This boring and boring R2-1 were the first borings made using the new sampling method. Relative densities were determined on four of the nine samples recovered. The values of relative density varied between 50 per cent to greater than 100 per cent in three of the samples and between 0 to 50 per cent in the other (sample 22 at 16.2 to 18.7 ft msl). The low values of relative density in the latter sample occurred at the top of the sample and values increased with the depth. The increased density may have been caused by pile action during the drive or the lower densities may be due to lignite, either is entirely feasible. However, even though lignite was visible in the sample, it did not appear to be present in the same proportions as in the samples described in boring R2-3 and appeared to be evenly distributed throughout the entire sample. This boring was stopped at a depth of 110 ft (elev -48 ft msl) for the same reasons as in boring R2-3. The locations where this boring was made caved into the river on 18 June.

Boring R2-1

13. Boring R2-1 was located 97 ft on the landside of the base line on range 2-D. The detailed log of the boring is shown on plate A7. The overburden soils existed to a depth of 42 ft (elev +22 ft msl). The top 6 ft consisted of lean clay underlain by 16 ft of fine brown sand, underlain in turn by 22 ft of silty soils. One sample (sample 4, elev 50.4 to 47.9 ft msl) was used for relative density determinations of the fine brown sand. The relative densities varied from -74 per cent to +125

per cent. The unusually low relative densities could be due to a separation of the sample in the tube resulting in a void space and consequently a greater volume than was actually occupied by the sand. The high relative densities are probably due to the method used for maximum density determinations in the laboratory.

14. Sixteen Shelby tube samples of the clean sand were obtained. One sample of silty sand was obtained at elev -70 ft msl. Full penetration of the sampler could not be obtained for 8 drives made in the clean sands below elev -25 ft msl. The samples below this elevation seemed to be very dense, both in appearance and in resistance to sampling, but densities could not be determined since the samples were too short. Three samples (19, 20, and 22) of the remaining 8 samples were used for density determinations. The relative densities averaged about 75 per cent for sample 19, 30 per cent for sample 20, and 50 per cent for sample 22.

Boring R2-4

15. Boring R2-4 was located 217 ft on the landside of the base line on range 2-D. The detailed log of the boring is shown on plate A8. The overburden soils were predominantly silts and existed to a depth of 44 ft (elev +20 ft msl) and were sampled readily at low values of penetration resistance.

16. Fourteen Shelby tube samples were taken in the clean sands. One sample (32) of silty sand was obtained at an elevation of -20 ft msl and two samples which contained clay, 42 and 48, were obtained at elevations of -40 and -60 ft. Relative density determinations were made on six of the 13 samples obtained in the clean sands. The relative densities

generally varied between 50 and 100 per cent. Values lower than 50 per cent were obtained in samples 26 and 38 at elevations of approximately -4 and -34 ft msl, respectively. These points, however, are rather localized and may represent very thin strata or may have been caused by the presence of a natural cavity or by a separation of the sample in the tube. Separation of the sample could occur during density determinations when the tube was held in an upright position, or during sampling due to a vacuum between the top of the sample and the stationary piston. This latter case is much less probable, as only one of the many sliced samples examined showed evidence of such separation. Sample 46 is the only known example of the latter condition. The length of the sample has been measured as 2.1 ft in the field. The ends of the sample were sealed with paraffin and the sample was shipped to the laboratory. The paraffin seal was still intact when the sample was opened; however, there was an 8-in. space between the end of the sample and the paraffin seal. The sample appeared to be in two separate portions; the lower 0.8 ft of the sample was badly disturbed, but the upper portion appeared to be relatively intact.

17. Two sand samples were used for permeability tests. The coefficients of vertical permeability for these samples were $k_v = 5 \times 10^{-4}$ cm per sec for sample 36, which had an over-all relative density of 81 per cent, and $k_v = 26 \times 10^{-4}$ for sample 44 which had an over-all relative density of 37 per cent. The tubes were opened and the samples inspected for disturbance before the dry densities were determined. Neither appeared to be disturbed.

Borings on Range 1-U

Boring R1U-1

18. Boring R1U-1 was located 240 ft on the riverside of the base line on range 1-U. The detailed log of this boring is shown on plate A9. The overburden soils existed to a depth of 65 ft (elev -5 ft msl). These soils consisted of silts, sandy-silts, silty-sands, and some clays. The cohesion varied from 0.35 to 0.5 ton per sq ft. Sample 17, a sliced sample, had shear planes which appeared to have been in the sample prior to sampling and to be due to natural causes.

19. Eight Shelby tube samples were taken in the clean sands. The relative densities of five of these samples varied from 50 to 100 per cent and averaged about 75 per cent. The lower relative densities appeared to be due to lignite.

Boring R1U-2

20. Boring R1U-2 was located 113 ft on the riverside of the base line on an offset 30 ft downstream of range 1-U. The detailed log of this boring is shown on plate A10. The overburden soils were predominantly silts and existed to a depth of 30 ft (elev +32 ft msl). The cohesion C_u varied from 0.2 to 0.7 ton per sq ft.

21. Nineteen Shelby tube samples of the clean sands were obtained. The relative densities varied from 0 to 100 per cent. Negative values were obtained in specimens of samples 19 and 21 which were due to lignite. The remainder of the low densities probably represent thin strata of relatively loose materials.

Boring R1U-3

22. Boring R1U-3 was located on the intersection of the base line and range 1-U. The detailed log of the boring is shown on plate A11. The overburden soils were predominantly silty, existing to a depth of 42 ft (elev +20 ft msl). A brown silty sand stratum occurred at elev 33 to 44 ft msl. Relative densities were determined for this material; they varied between 25 and 75 per cent. Relative densities were also determined for a stratum of clean fine sand which occurred in the overburden (sample 14, elev 23.4 to 25.9 ft msl). The relative densities generally varied between 25 and 50 per cent, one very low value being due to lignite.

Boring R1U-4

23. Boring R1U-4 was located 150 ft on the landside of the base line on an offset 40 ft upstream of range 1-U. The detailed log of this boring is shown on plate A12. The overburden soils were predominantly silt, existing to a depth of 33 ft (elev 31 ft msl). These soils were generally soft, the cohesion, C_u, varying from 0.2 to 0.3 ton per sq ft.

24. Twenty Shelby tube samples of the clean sands were taken. The relative densities averaged about 50 per cent for five of the samples (14, 17, 19, 24, and 31) and about 75 per cent for two (22 and 25). The two samples nearest the surface (14 and 17 at elev 21 and 7 ft msl) and the deepest sample (31, at elev -52 ft msl) had specimens having the lowest relative densities.

Borings on Range 2-U

Boring R2U-1

25. Boring R2U-1 was located on range 2-U at the toe of the levee. The detailed log of this boring is shown on plate A13. The overburden soils existed to a depth of 43 ft (elev 13 ft msl). Lean clays and silts were predominant (C_u varying from 0.1 to 0.2 ton per sq ft).

26. The fine grey sands of the deep sand formation were encountered from 13 ft msl to -16 ft msl, where there was a 12-ft stratum of clay. The clay was underlain by the coarser sands typical of such depths. The sands above the lower clay stratum had relative densities averaging 75 to 100 per cent. The cohesion, C_u, of the lower clay stratum varied from 0.3 to 0.9 ton per sq ft; the stratum contained shells.

Boring R2U-2

27. Boring R2U-2 was located 1500 ft on the landside of the intersection of range 2-U and the toe of the levee on a line perpendicular to the toe of the levee. The detailed log of this boring is shown on plate A14. The overburden soils existed to a depth of 82 ft (elev -24 ft msl) and consisted principally of lean clays. The cohesion, C_u, of these soils varied from about 0.2 to 0.7 ton per sq ft.

28. Five Shelby tube samples of the clean sands were obtained. The relative densities of three of these samples averaged between about 40 and 80 per cent.

Top of Bank Borings

Boring R9U-1

29. Boring R9U-1 was located at the top of the bank on range 9-U. The detailed log of the boring is shown on plate A15. The overburden soils consisted principally of silts with some lean clays, to a depth of 65 ft. The silts were generally soft while the clays had a tendency to be rather stiff, the cohesion, C_u, being about 0.6 to 0.7 ton per sq ft.

30. Five Shelby tube samples were obtained in the clean sands. Relative densities were determined on two of these samples and averaged about 75 per cent.

Boring R3U-1

31. Boring R3U-1 was located 254 ft on the riverside of the base line on range 3-U. The detailed log of this boring is shown on plate A16. The overburden soils were predominantly silt and existed to a depth of 55 ft (elev +4 ft msl).

32. Nine Shelby tube samples were taken in the clean sands. Relative densities were determined on five of these samples and varied from 50 to 100 per cent.

Borings R10-2 and R10-2A

33. Borings R10-2 and R10-2A were located 340 ft on the landside of the base line on range 10; boring R10-2A was offset 20 ft upstream from range 10. Boring R10-2 was the first made in this investigation and the improved method of sampling had not been developed at this time.

The detailed logs of these borings are shown on plates A17 and A18. Boring R10-2 was made in a cased bore hole filled with water. Samples could not be obtained in the clean sands by this method. Boring R10-2A was made (by the method described previously using drilling mud) to supplement boring R10-2. The relative density of two samples of the overburden silty sands varied between 25 and 75 per cent. The relative densities of the clean sand samples averaged about 75 per cent.

Boring R20-1

34. Boring R20-1 was located 212 ft on the landside of the base line on range 20-D. The detailed log of this boring is shown on plate A19. Overburden soils were predominantly silty-sands and sands. These soils existed to a depth of 42 ft (elev 22 ft msl). Clay strata existed near the surface but were very dry and had greater cohesion in the remolded state (C_m varying 0.7 to 0.9) than in the undisturbed state (C_u varying 0.4 to 0.5).

35. Fourteen Shelby tube samples were obtained in the clean sands. Relative densities determined on four of these samples averaged about 50 per cent.

Boring R30-1

36. Boring R30-1 was located 67 ft on the riverside of the base line on range 30-D. The detailed log of this boring is shown on plate A20. The overburden soils were predominantly silts and existed to a depth of 27 ft (elev +35 ft msl). One stratum of fine brown sand existed between elev 26 to 34 ft msl. It had a relative density varying between

50 to 75 per cent.

37. Twelve Shelby tube samples were obtained in the clean sands. The relative density of 7 of these samples varied between 50 and 75 per cent, the majority being at or near 75 per cent. Sample 14 (elev 8.8 to 6.3 ft msl) had a relative density averaging about 50 per cent.

Boring R34-1

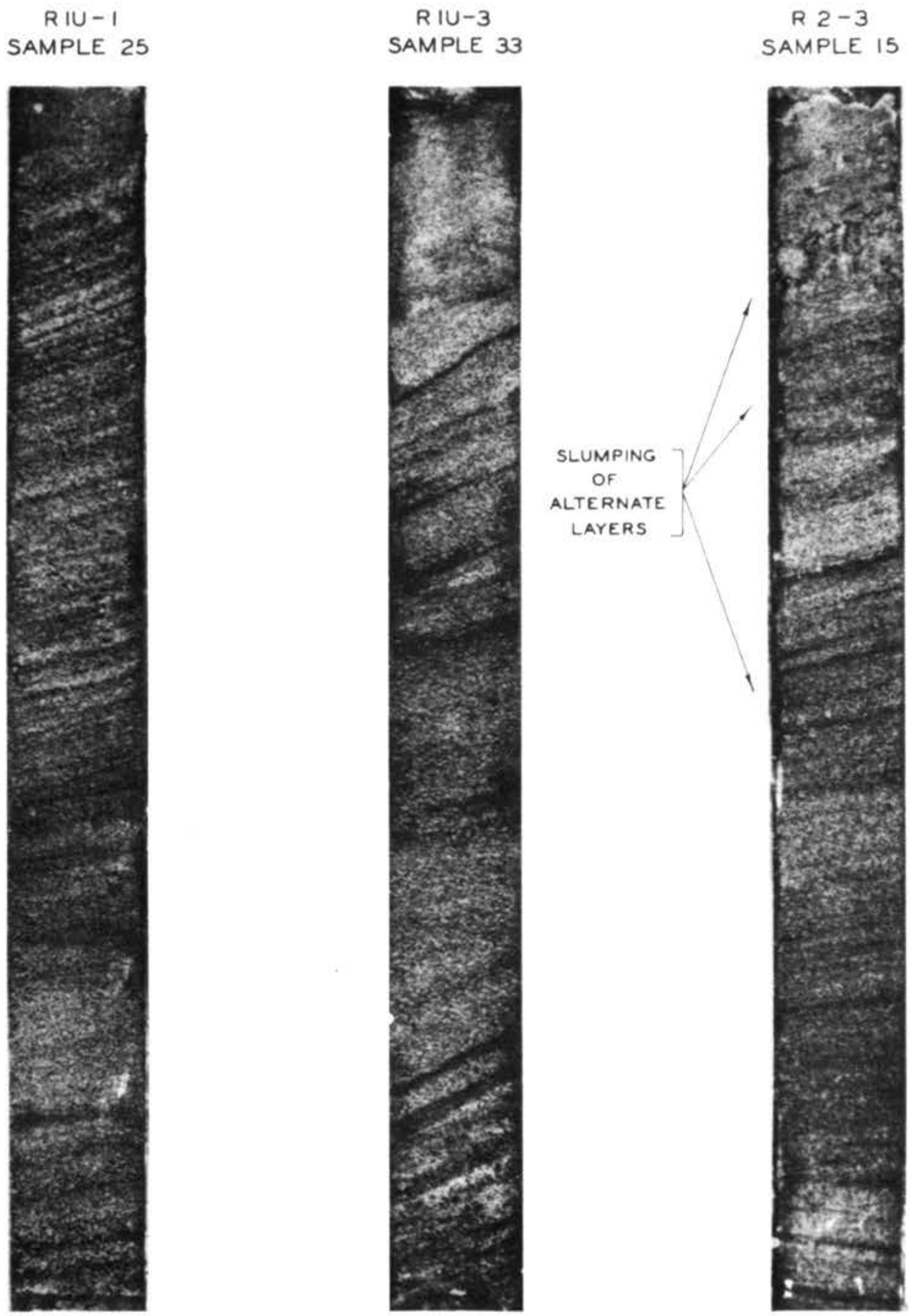
38. Boring R34-1 was located 135 ft on the riverside of the base line on range 34-D. The detailed log of this boring is shown on plate A21. The overburden soils were predominantly silt, existing to a depth of 28 ft (elev +33 ft msl).

39. Fourteen Shelby tube samples were obtained in the clean sands The relative densities of six of these samples varied from 0 to 75 per cent, the average relative density being about 50 per cent.

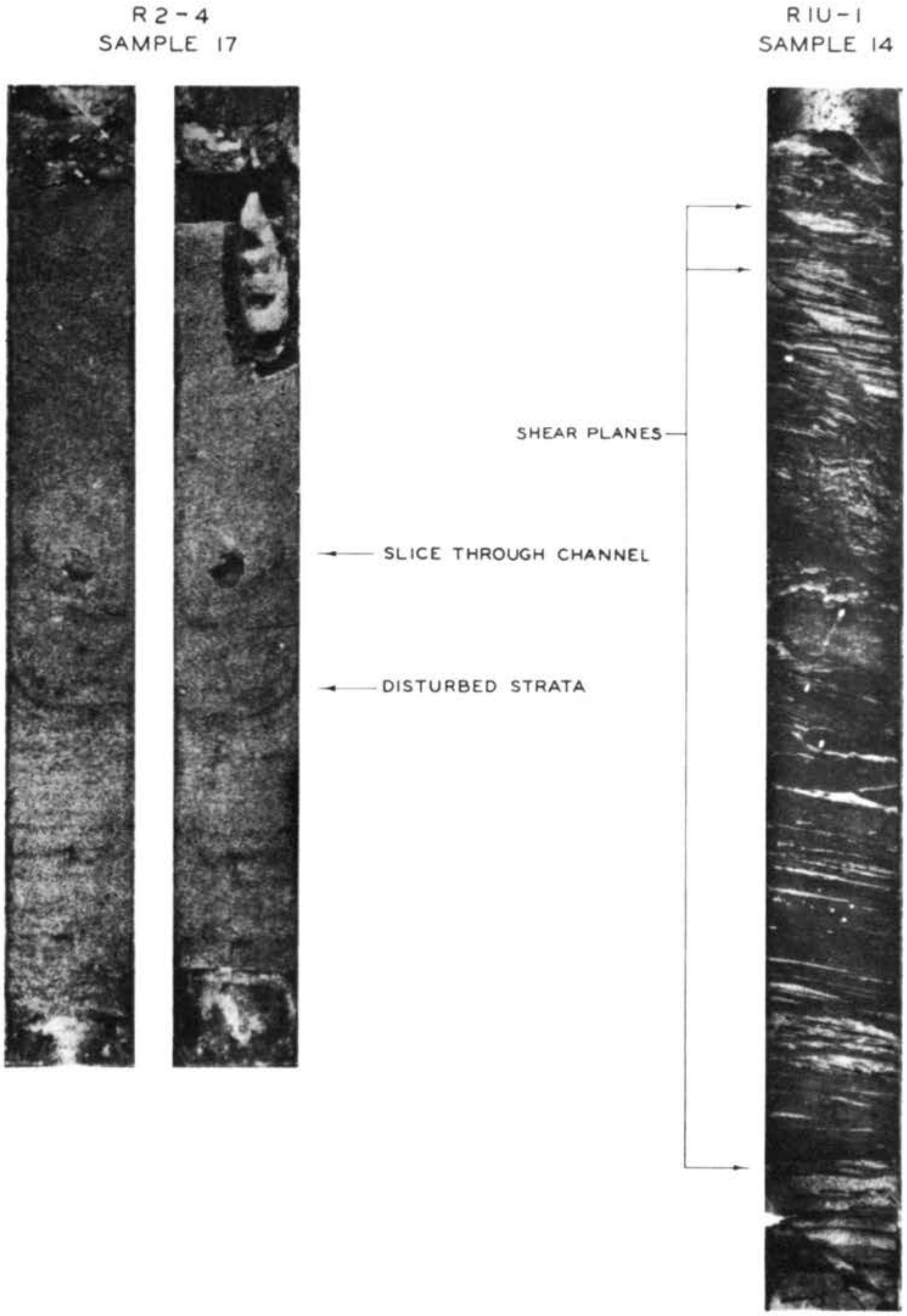
Piezometer Observations

40. The river stood at about 40 ft msl during the first week (2 to 9 July 1949) after installation of the piezometers, and all of the piezometers indicated that the hydrostatic pressure in the clean sands was equal to or slightly less than that caused by a head of water equal to the elevation of the river. During the second week (9-16 July) the river gradually dropped to about elev 32.5 ft msl, while the piezometer readings dropped to about 33 to 34 ft msl. The maximum river-piezometer differential during this period was 1.5 ft at range R-20-D. From 17 to 26 July the river rose to elev 38.8 ft msl. All of the piezometers indicated piezometric levels slightly above that of the river during this

PLATES



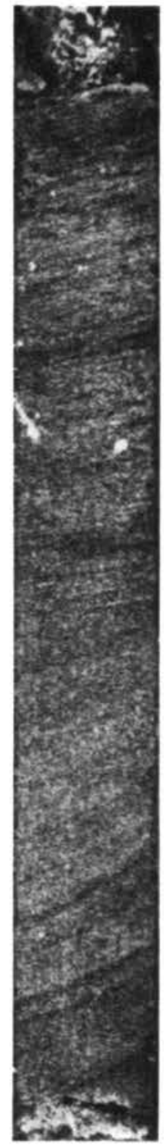
PHOTOGRAPHS OF SAMPLES
R IU-1 SAMPLE 25
R IU-3 SAMPLE 33
R 2-3 SAMPLE 15



PHOTOGRAPHS OF SAMPLES
R 2-4 SAMPLE 17
R IU-I SAMPLE 14

R 1U-4
SAMPLE 16

R 2-2
SAMPLE 8

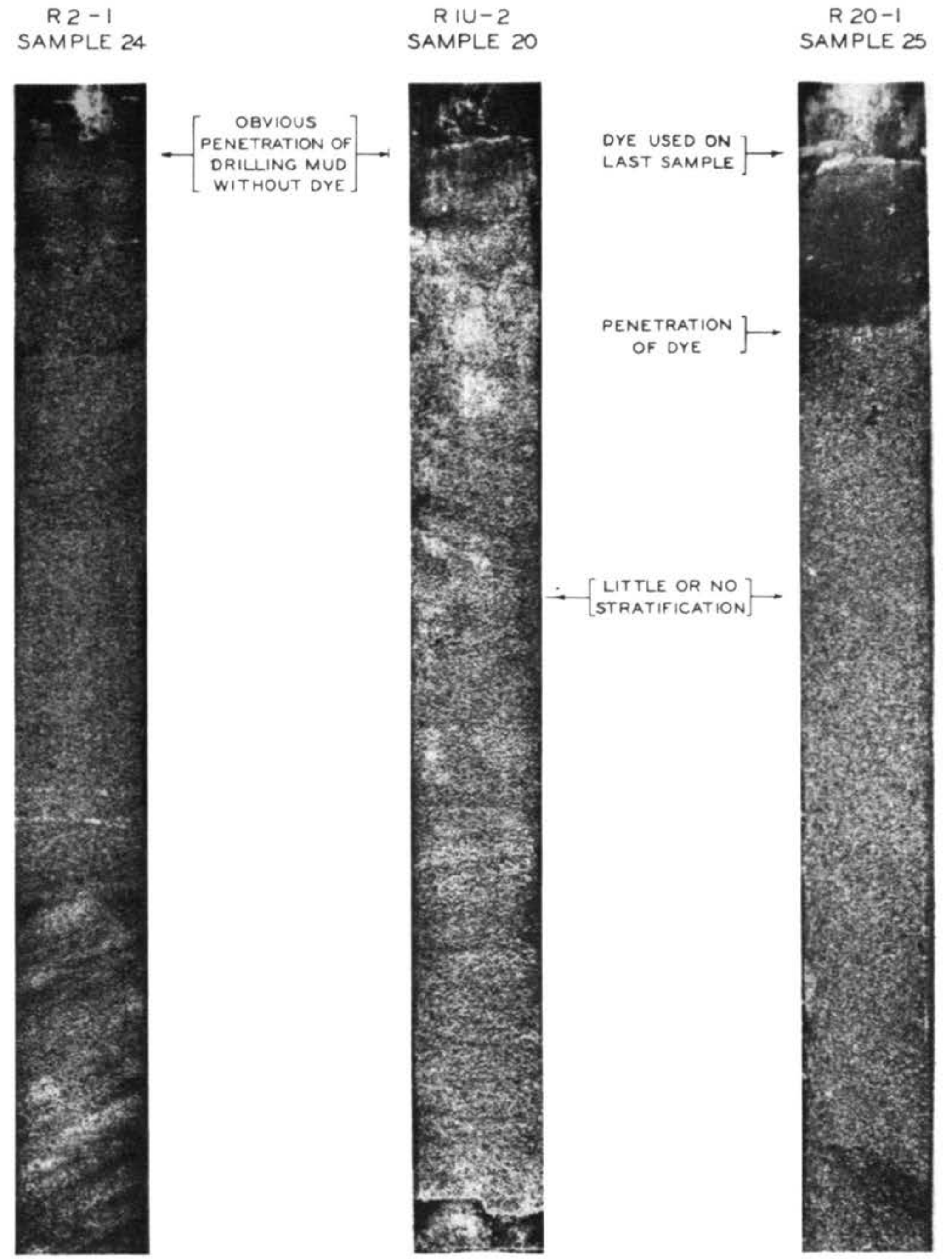


NO OBVIOUS PENETRATION OF DRILLING MUD

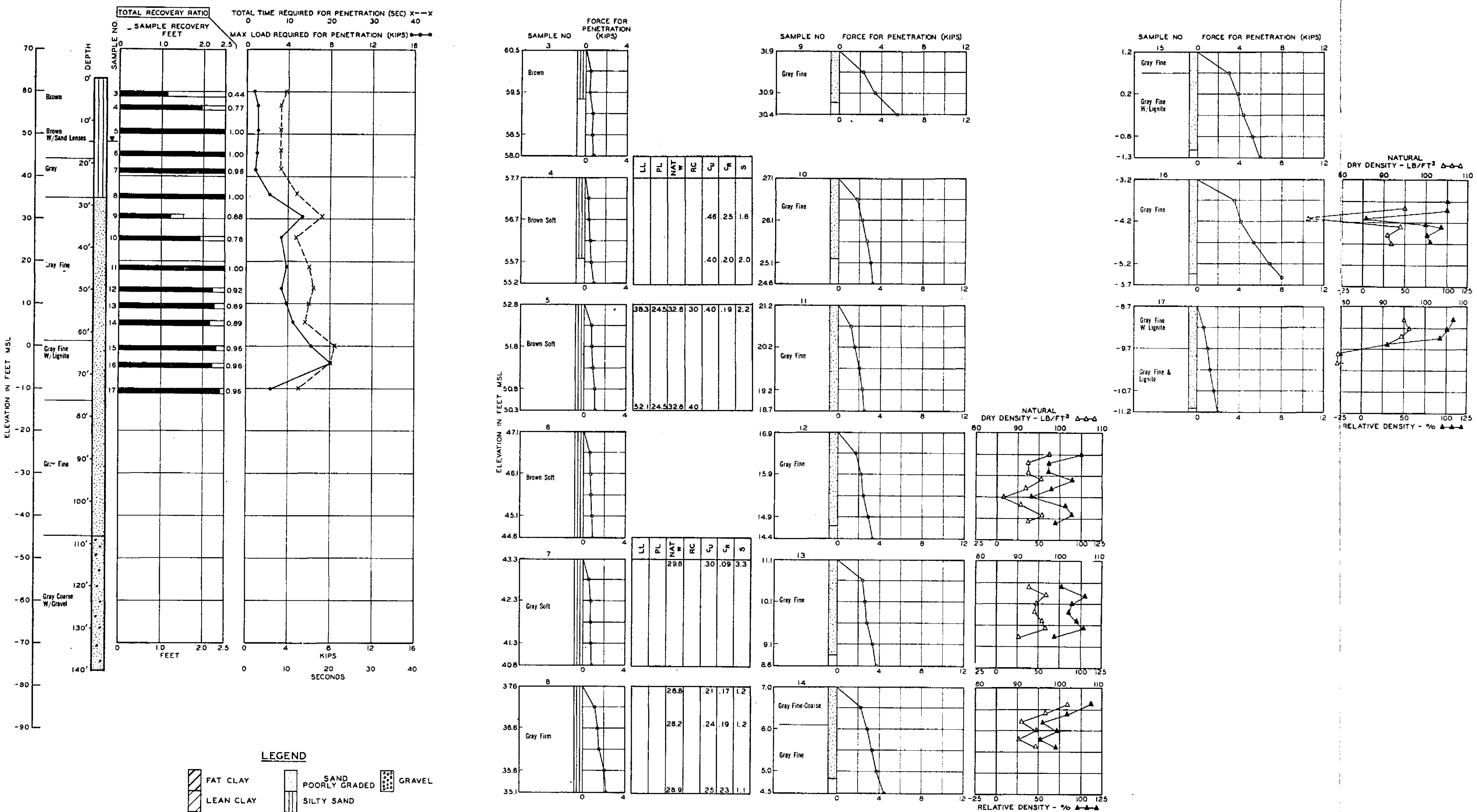
LITTLE TO NO STRATIFICATION

PHOTOGRAPHS OF SAMPLES

R 1U-4 SAMPLE 16
R 2-2 SAMPLE 8



PHOTOGRAPHS OF SAMPLES
R 2 - 1 SAMPLE 24
R 1U- 2 SAMPLE 20
R 20- 1 SAMPLE 25



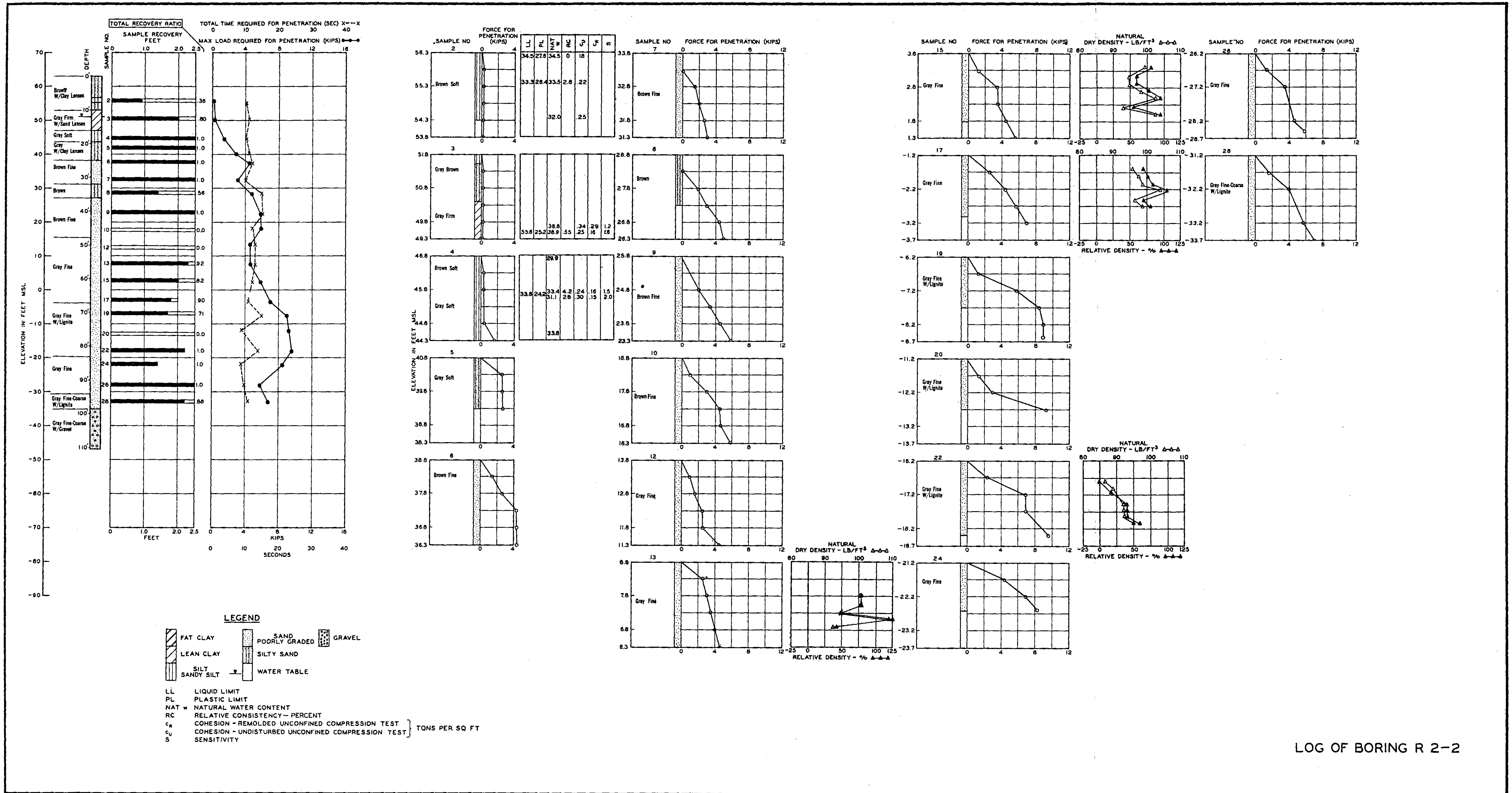
LEGEND

FAT CLAY
LEAN CLAY
SILT SANDY SILT
SAND POORLY GRADED
SILTY SAND
WATER TABLE
GRAVEL

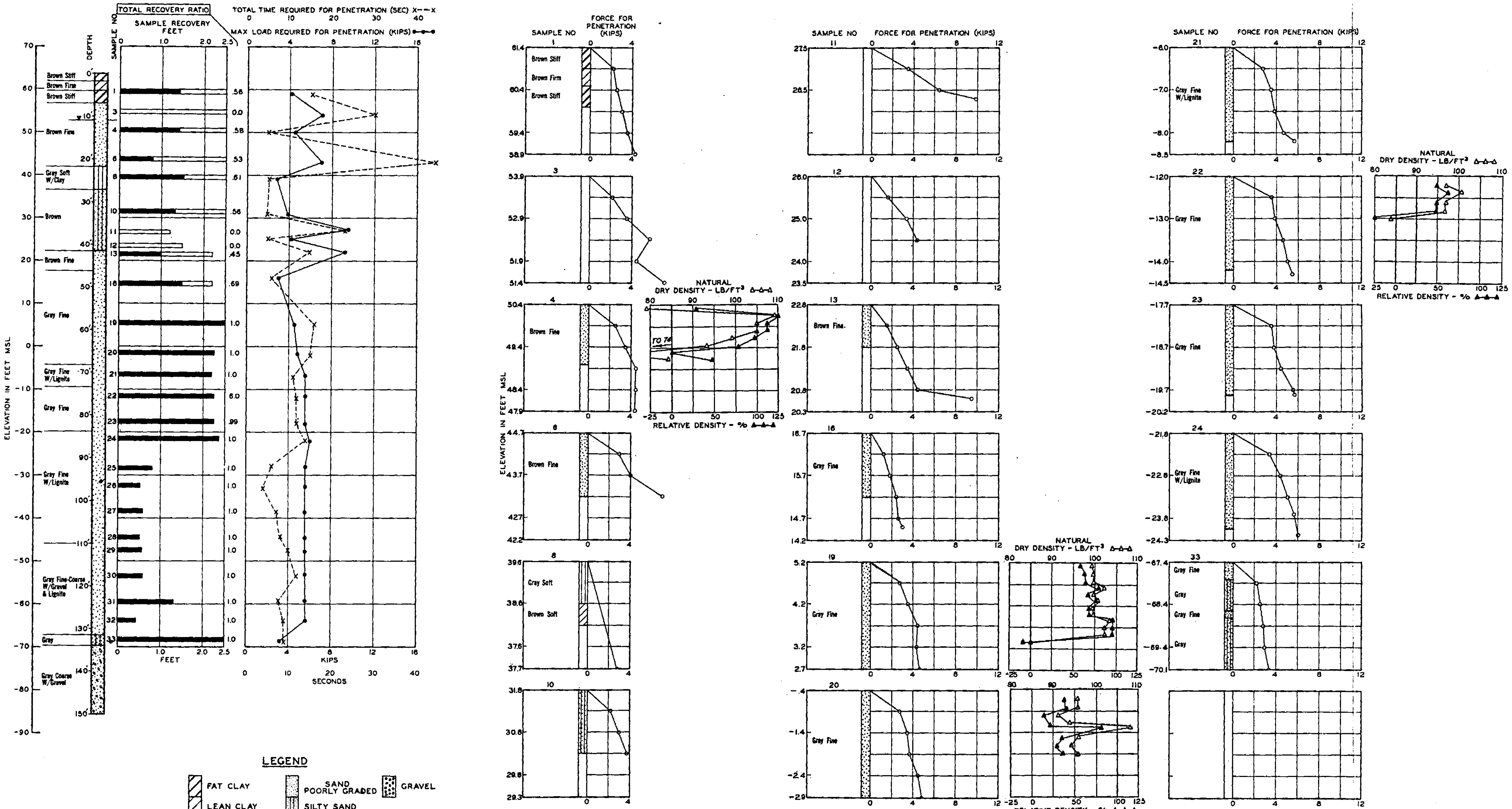
LL LIQUID LIMIT
PL PLASTIC LIMIT
NAT w NATURAL WATER CONTENT
RC RELATIVE CONSISTENCY — PERCENT
c_R COHESION - REMOLDED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
c_u COHESION - UNDISTURBED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
S SENSITIVITY

LOG OF BORING R2-3

PLATE A5



TOTAL RECOVERY RATIO
SAMPLE RECOVERY FEET
TOTAL TIME REQUIRED FOR PENETRATION (SEC) X--X
MAX LOAD REQUIRED FOR PENETRATION (KIPS)
ELEVATION IN FEET MSL
DEPTH
SAMPLE NO.
FEET
KIPS
SECONDS
FORCE FOR PENETRATION (KIPS)
NATURAL DRY DENSITY - LB/FT³
RELATIVE DENSITY - %
LEGEND
FAT CLAY
LEAN CLAY
SILT SANDY SILT
SAND POORLY GRADED
SILTY SAND
WATER TABLE
GRAVEL
LL LIQUID LIMIT
PL PLASTIC LIMIT
NAT w NATURAL WATER CONTENT
RC RELATIVE CONSISTENCY— PERCENT
c_R COHESION - REMOLDED UNCONFINED COMPRESSION TEST
c_U COHESION - UNDISTURBED UNCONFINED COMPRESSION TEST
TONS PER SQ FT
S SENSITIVITY
LOG OF BORING R 2-2

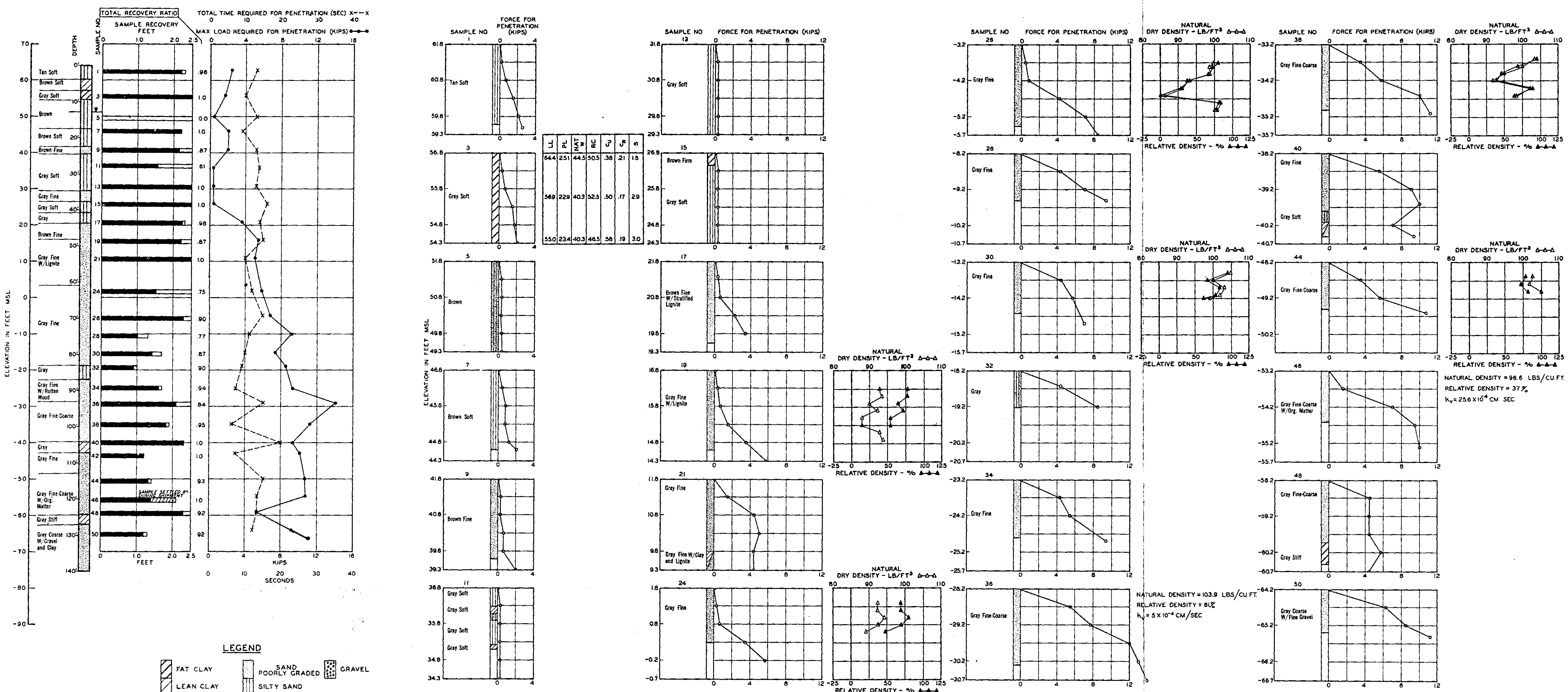


LEGEND

FAT CLAY
LEAN CLAY
SILT SANDY SILT
SAND POORLY GRADED
SILTY SAND
WATER TABLE
GRAVEL

LL LIQUID LIMIT
PL PLASTIC LIMIT
NAT w NATURAL WATER CONTENT
RC RELATIVE CONSISTENCY—PERCENT
c_R COHESION - REMOLDED UNCONFINED COMPRESSION TEST } TONS PER SQ FT.
c_U COHESION - UNDISTURBED UNCONFINED COMPRESSION TEST } TONS PER SQ FT.
S SENSITIVITY

LOG OF BORING R2-1



LL	PL	NAT w	RC	c_u	c_R	S
64.4	25.1	44.5	50.5	.38	.21	1.8
56.9	22.9	40.3	52.5	.50	.17	2.9
55.0	23.4	40.3	46.5	.58	.19	3.0

NATURAL DENSITY = 103.9 LBS/CU.FT.
RELATIVE DENSITY = 81%
k_v = 5 X 10^{-4} CM/SEC

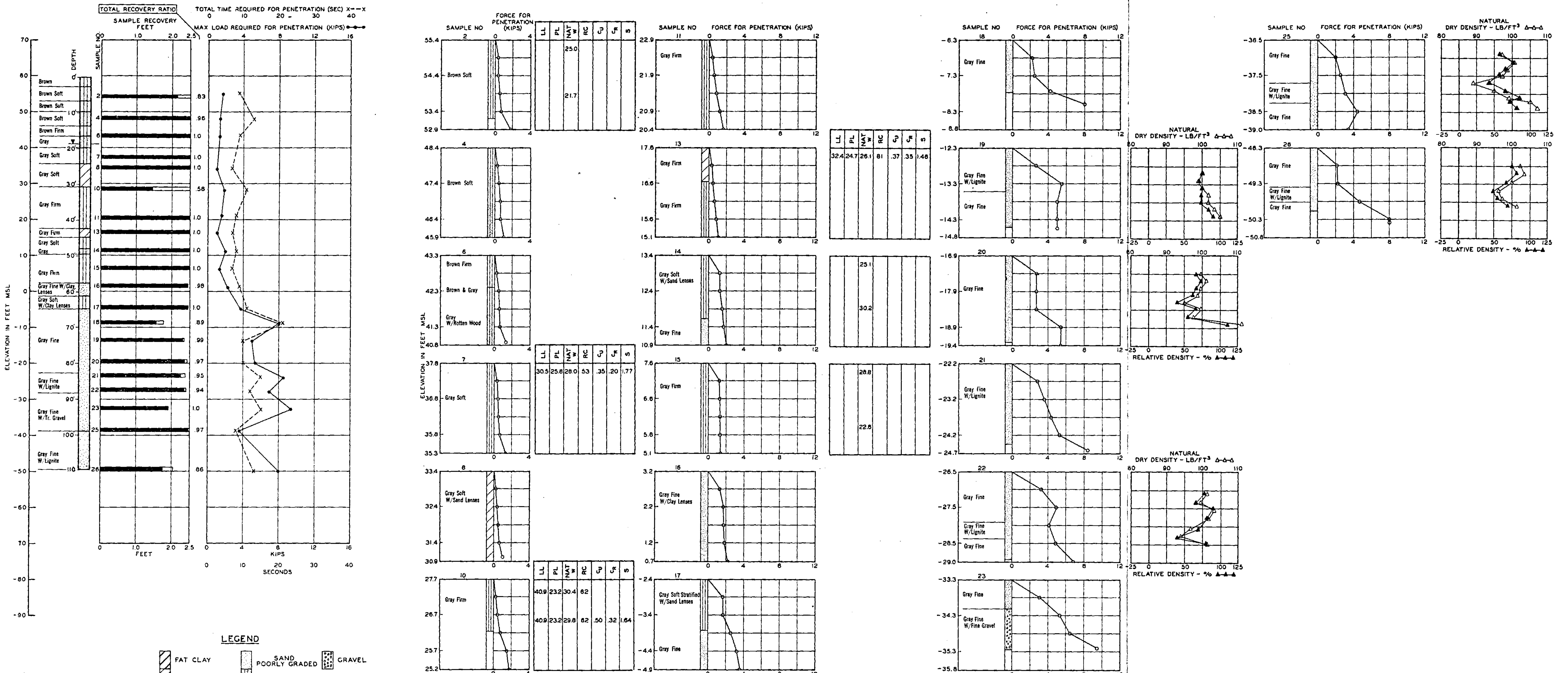
NATURAL DENSITY = 96.6 LBS/CU.FT.
RELATIVE DENSITY = 37%
k_v = 25.6 X 10^{-4} CM SEC

LEGEND

- FAT CLAY
- LEAN CLAY
- SILT SANDY SILT
- SAND POORLY GRADED
- SILTY SAND
- WATER TABLE
- GRAVEL

LL LIQUID LIMIT
PL PLASTIC LIMIT
NAT w NATURAL WATER CONTENT
RC RELATIVE CONSISTENCY - PERCENT
c_R COHESION - REMOLDED UNCONFINED COMPRESSION TEST } TONS PER SQ.FT.
c_u COHESION - UNDISTURBED UNCONFINED COMPRESSION TEST } TONS PER SQ.FT.
S SENSITIVITY

LOG OF BORING R2-4

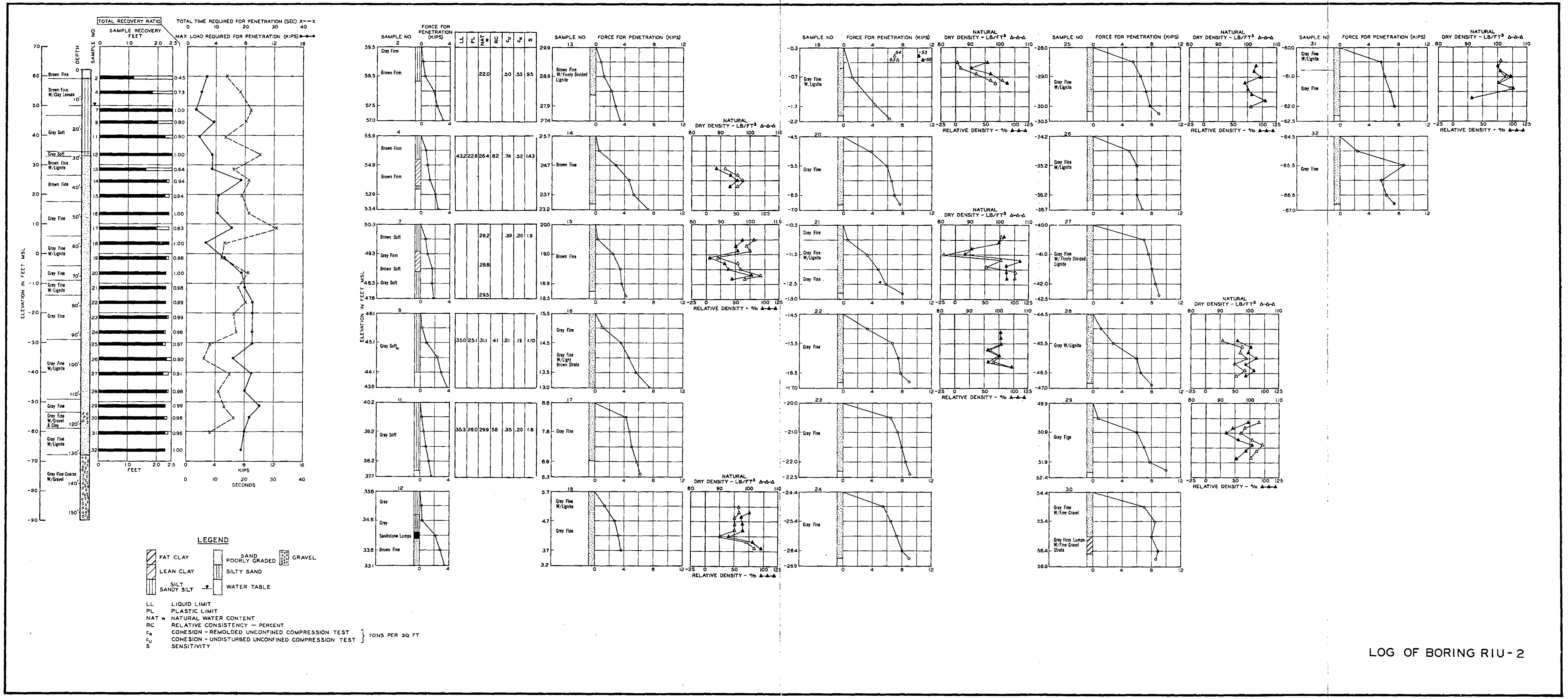


LEGEND

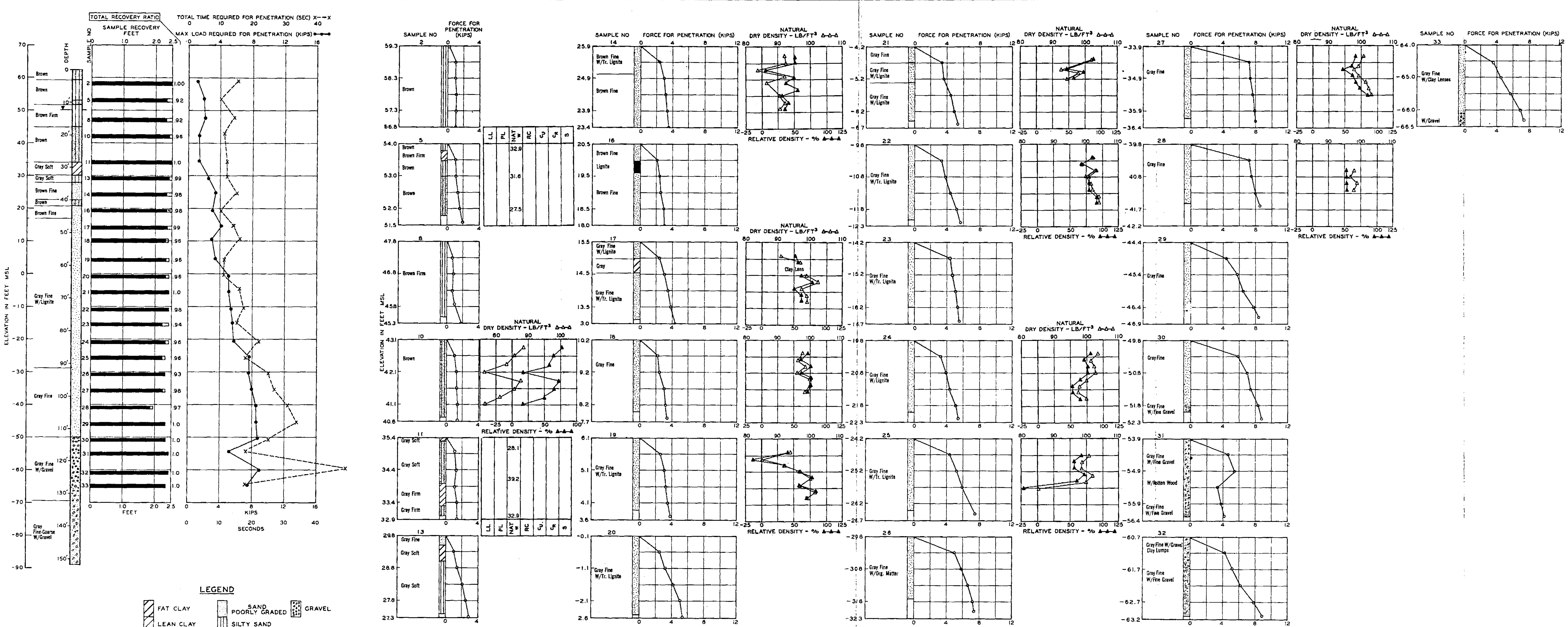
- FAT CLAY
- LEAN CLAY
- SILT SANDY SILT
- SAND POORLY GRADED
- SILTY SAND
- WATER TABLE
- GRAVEL

LL. LIQUID LIMIT
PL PLASTIC LIMIT
NAT w NATURAL WATER CONTENT
RC RELATIVE CONSISTENCY - PERCENT
c_R COHESION - REMOLDED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
c_U COHESION - UNDISTURBED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
S SENSITIVITY

LOG OF BORING R1U-1



LOG OF BORING R1U-2

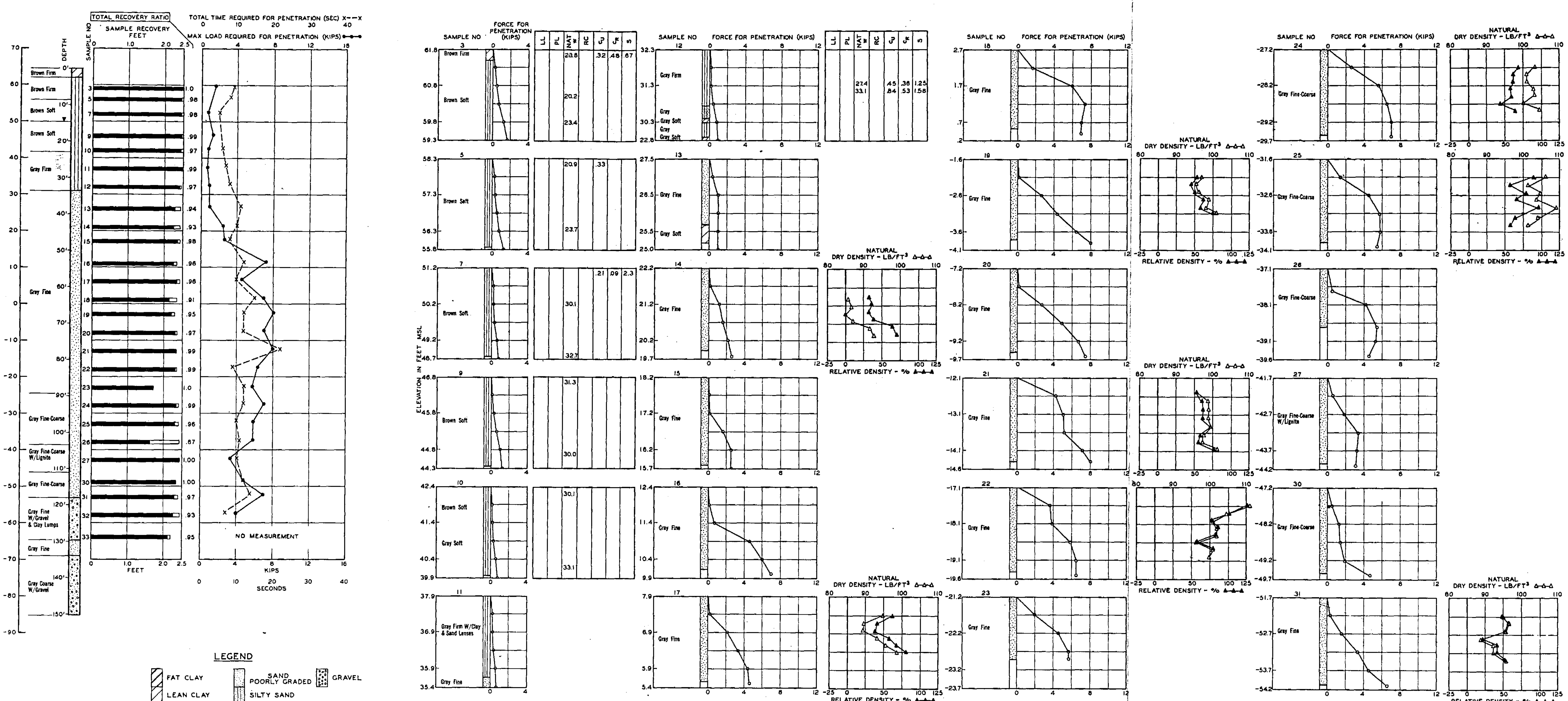


LEGEND

- FAT CLAY
- LEAN CLAY
- SILT / SANDY SILT
- SAND / POORLY GRADED
- SILTY SAND
- WATER TABLE
- GRAVEL

LL LIQUID LIMIT
PL PLASTIC LIMIT
NAT w NATURAL WATER CONTENT
RC RELATIVE CONSISTENCY - PERCENT
c_R COHESION - REMOLDED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
c_U COHESION - UNDISTURBED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
S SENSITIVITY

LOG OF BORING R IU-3

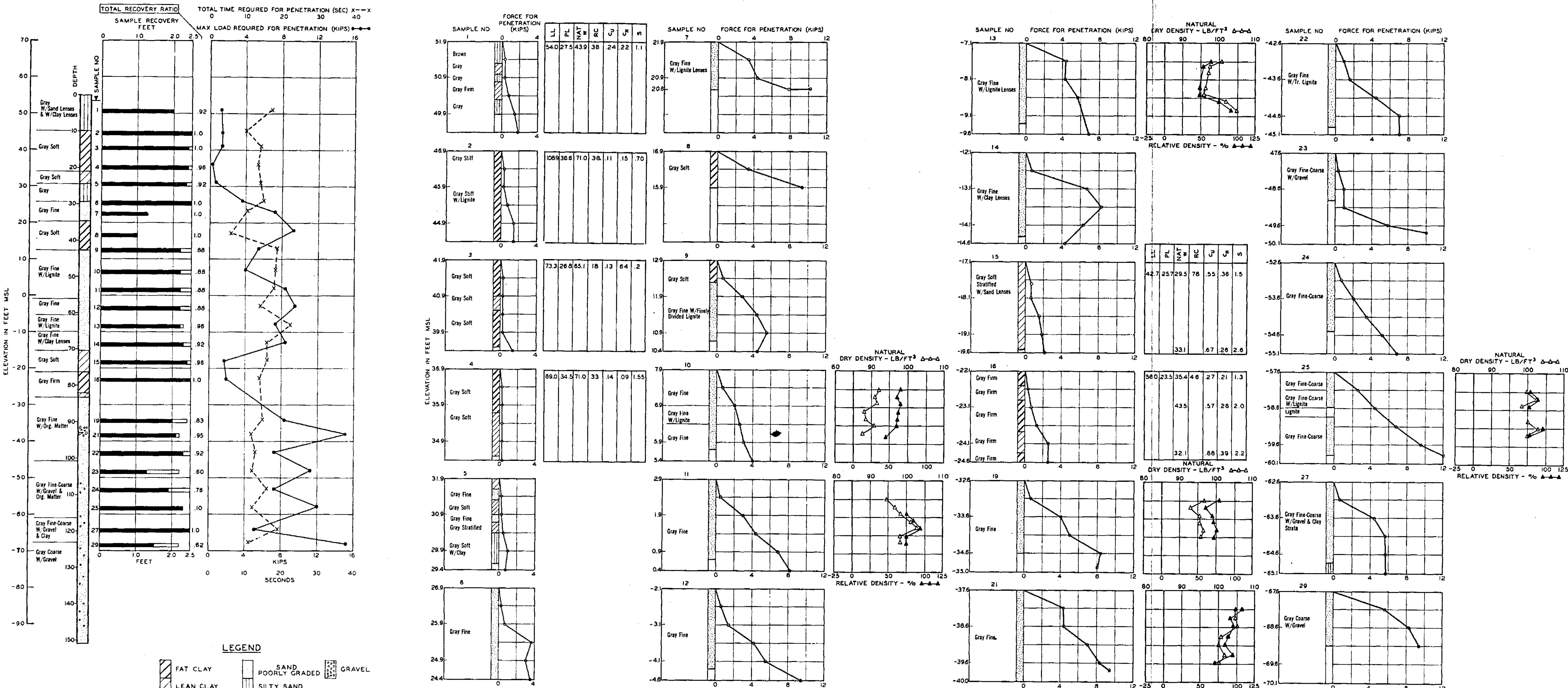


LEGEND

FAT CLAY
LEAN CLAY
SILT SANDY SILT
SAND POORLY GRADED
SILTY SAND
WATER TABLE
GRAVEL

LL LIQUID LIMIT
PL PLASTIC LIMIT
NAT w NATURAL WATER CONTENT
RC RELATIVE CONSISTENCY – PERCENT
c_R COHESION – REMOLDED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
c_U COHESION – UNDISTURBED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
S SENSITIVITY

LOG OF BORING R1U-4

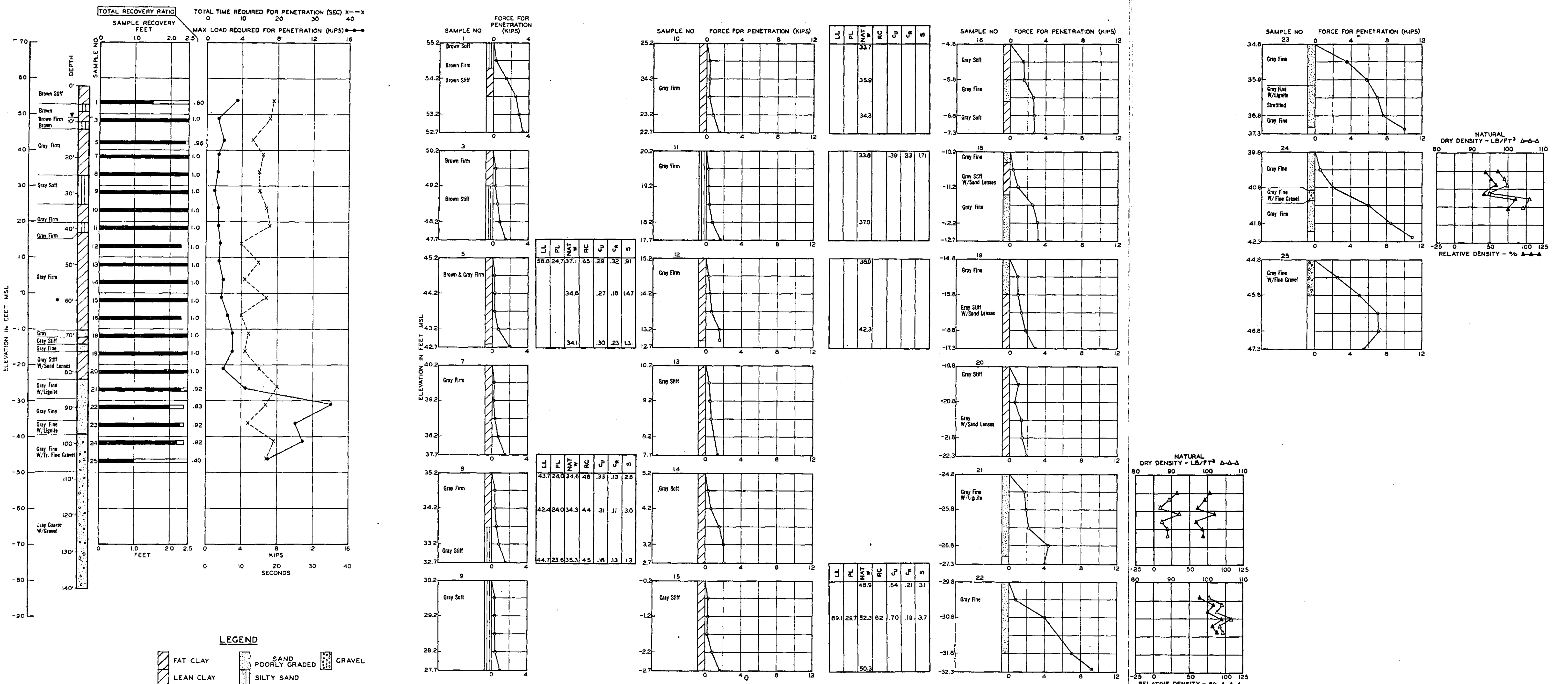


LEGEND

- FAT CLAY
- LEAN CLAY
- SILT / SANDY SILT
- SAND POORLY GRADED
- SILTY SAND
- WATER TABLE
- GRAVEL

LL	LIQUID LIMIT
PL	PLASTIC LIMIT
NAT w	NATURAL WATER CONTENT
RC	RELATIVE CONSISTENCY — PERCENT
c_R	COHESION — REMOLDED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
c_U	COHESION — UNDISTURBED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
S	SENSITIVITY

LOG OF BORING R 2U-1



LL	PL	NAT w	RC	c_U	c_R	S
58.8	24.7	37.1	.65	.29	.32	.91
		34.6		.27	.18	1.47
		34.1		.30	.23	1.3

LL	PL	NAT w	RC	c_U	c_R	S
43.7	24.0	34.6	46	.33	.13	2.6
42.4	24.0	34.3	44	.31	.11	3.0
44.7	23.6	35.3	45	.18	.13	1.3

LL	PL	NAT w	RC	c_U	c_R	S
		33.7				
		35.9				
		34.3				
		33.8		.39	.23	1.71
		37.0				
		38.9				
		42.3				

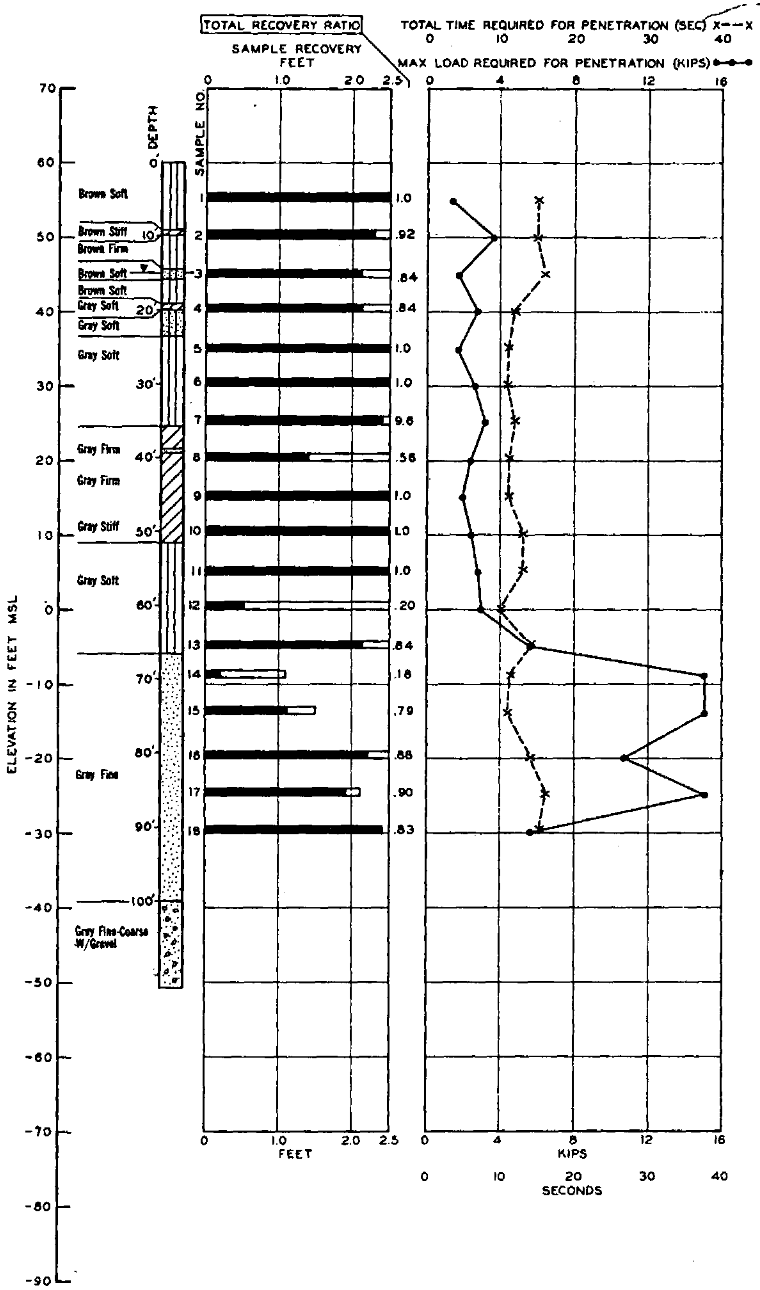
LL	PL	NAT w	RC	c_U	c_R	S
		48.9		.64	.21	3.1
89.1	29.7	52.3	62	.70	.19	3.7
		50.3				

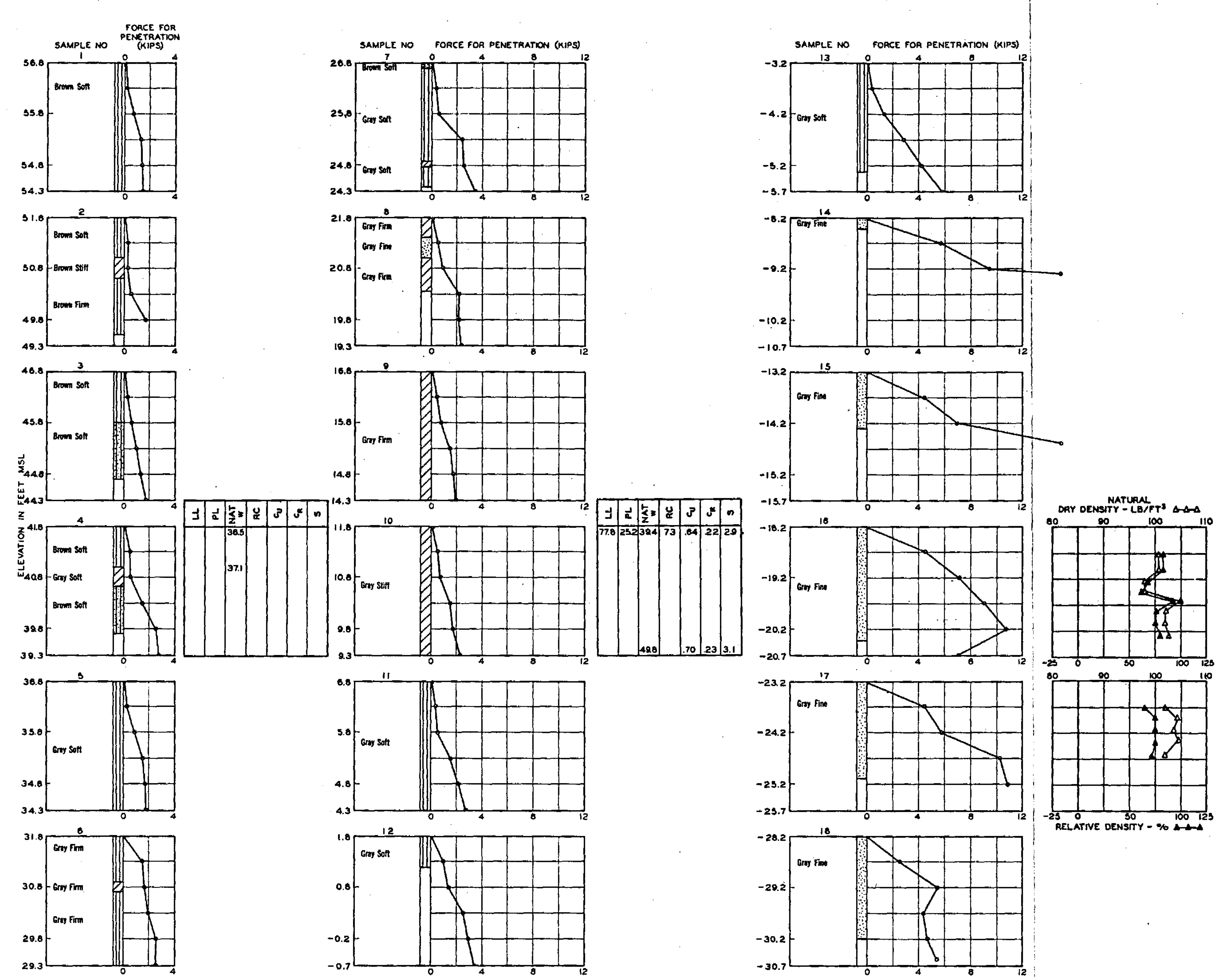
LEGEND

- FAT CLAY
- LEAN CLAY
- SILT, SANDY SILT
- SAND, POORLY GRADED
- SILTY SAND
- WATER TABLE
- GRAVEL

LL — LIQUID LIMIT
PL — PLASTIC LIMIT
NAT w — NATURAL WATER CONTENT
RC — RELATIVE CONSISTENCY - PERCENT
c_R — COHESION - REMOLDED UNCONFINED COMPRESSION TEST (TONS PER SQ FT)
c_U — COHESION - UNDISTURBED UNCONFINED COMPRESSION TEST (TONS PER SQ FT)
S — SENSITIVITY

LOG OF BORING R 2U-2





LL	PL	NAT w	RC	c_U	c_R	S
		38.5				
		37.1				

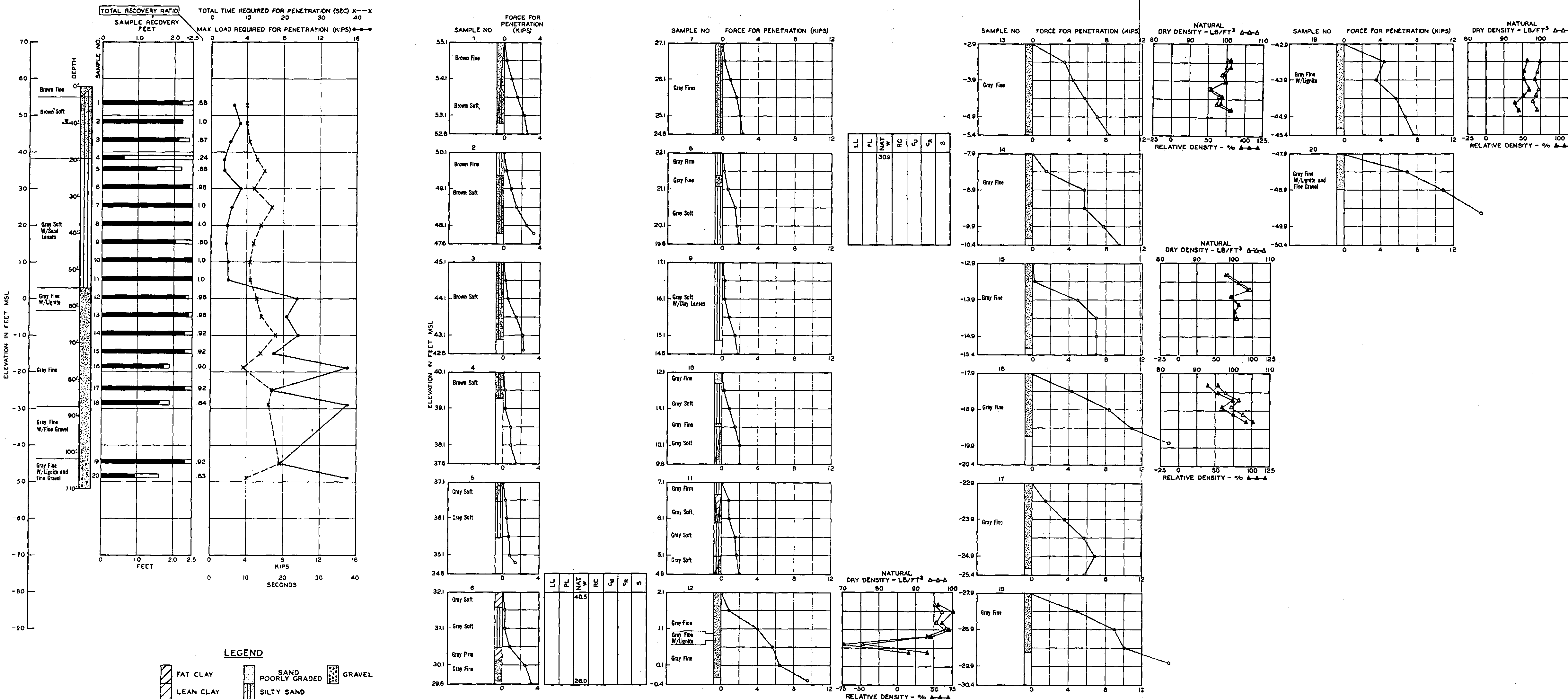
LL	PL	NAT w	RC	c_U	c_R	S
77.8	25.2	39.4	73	.64	.22	2.9
		49.8		.70	.23	3.1

LEGEND

- FAT CLAY
- LEAN CLAY
- SILT SANDY SILT
- SAND POORLY GRADED
- SILTY SAND
- WATER TABLE
- GRAVEL

LL LIQUID LIMIT
PL PLASTIC LIMIT
NAT w NATURAL WATER CONTENT
RC RELATIVE CONSISTENCY - PERCENT
c_R COHESION - REMOLDED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
c_U COHESION - UNDISTURBED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
S SENSITIVITY

LOG OF BORING R9U-1

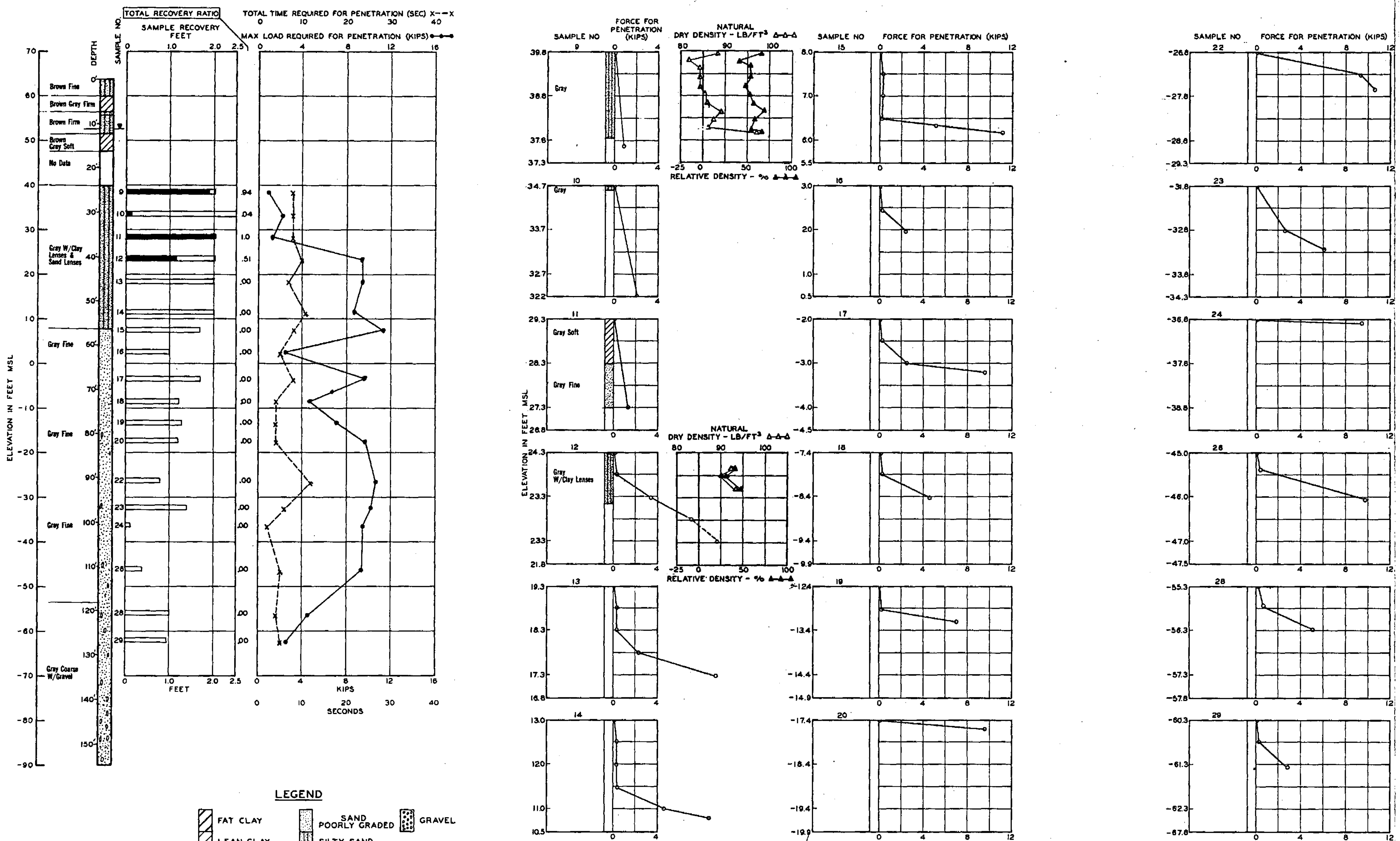


LEGEND

- FAT CLAY
- LEAN CLAY
- SILT SANDY SILT
- SAND POORLY GRADED
- SILTY SAND
- WATER TABLE
- GRAVEL

LL	LIQUID LIMIT
PL	PLASTIC LIMIT
NAT w	NATURAL WATER CONTENT
RC	RELATIVE CONSISTENCY - PERCENT
c_R	COHESION - REMOLDED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
c_U	COHESION - UNDISTURBED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
S	SENSITIVITY

LOG OF BORING R 3U-1

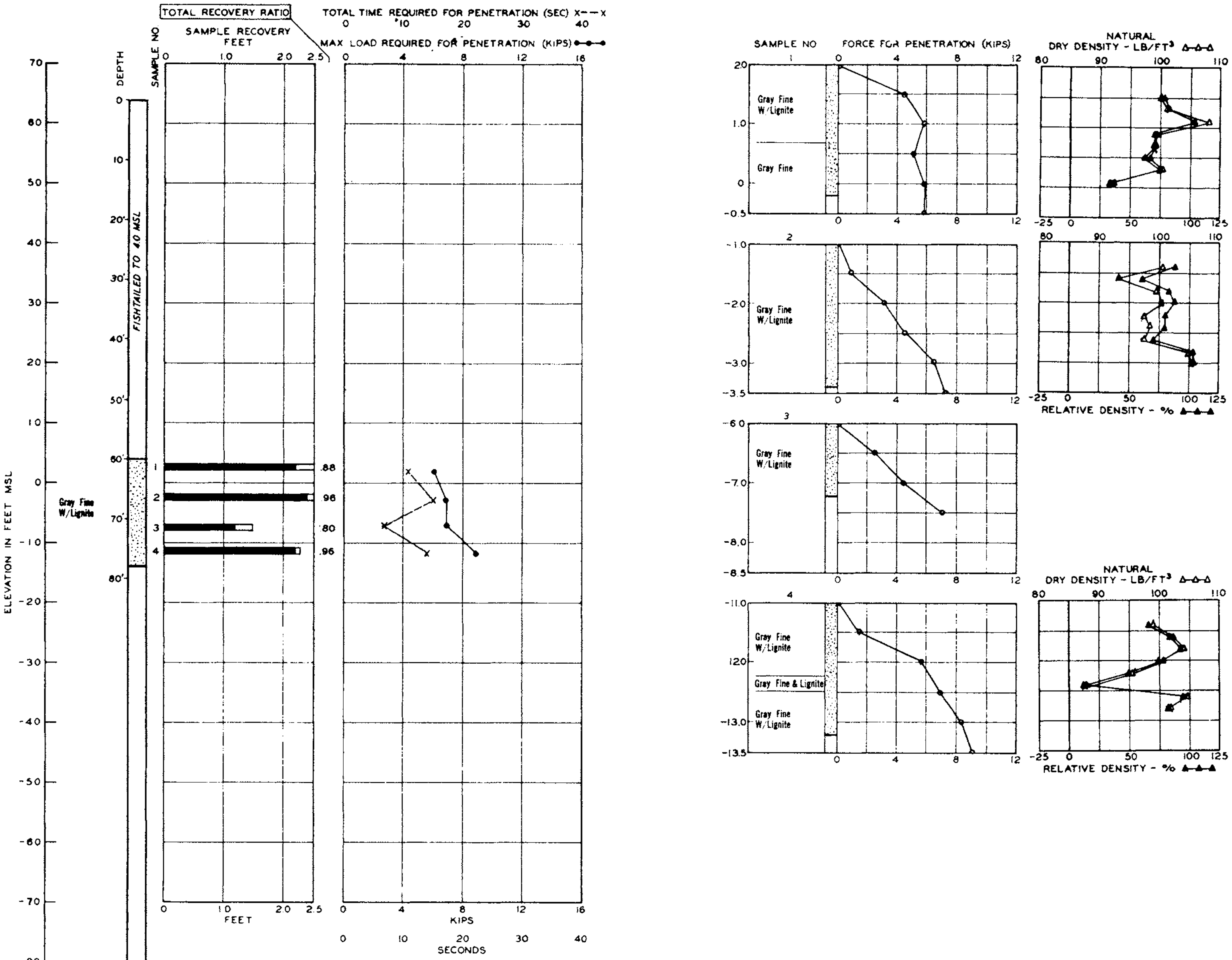


LEGEND

FAT CLAY
LEAN CLAY
SILT SANDY SILT
SAND POORLY GRADED
SILTY SAND
WATER TABLE
GRAVEL

LL	LIQUID LIMIT
PL	PLASTIC LIMIT
NAT w	NATURAL WATER CONTENT
RC	RELATIVE CONSISTENCY - PERCENT
c_R	COHESION - REMOLDED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
c_U	COHESION - UNDISTURBED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
S	SENSITIVITY

LOG OF BORING R10-2

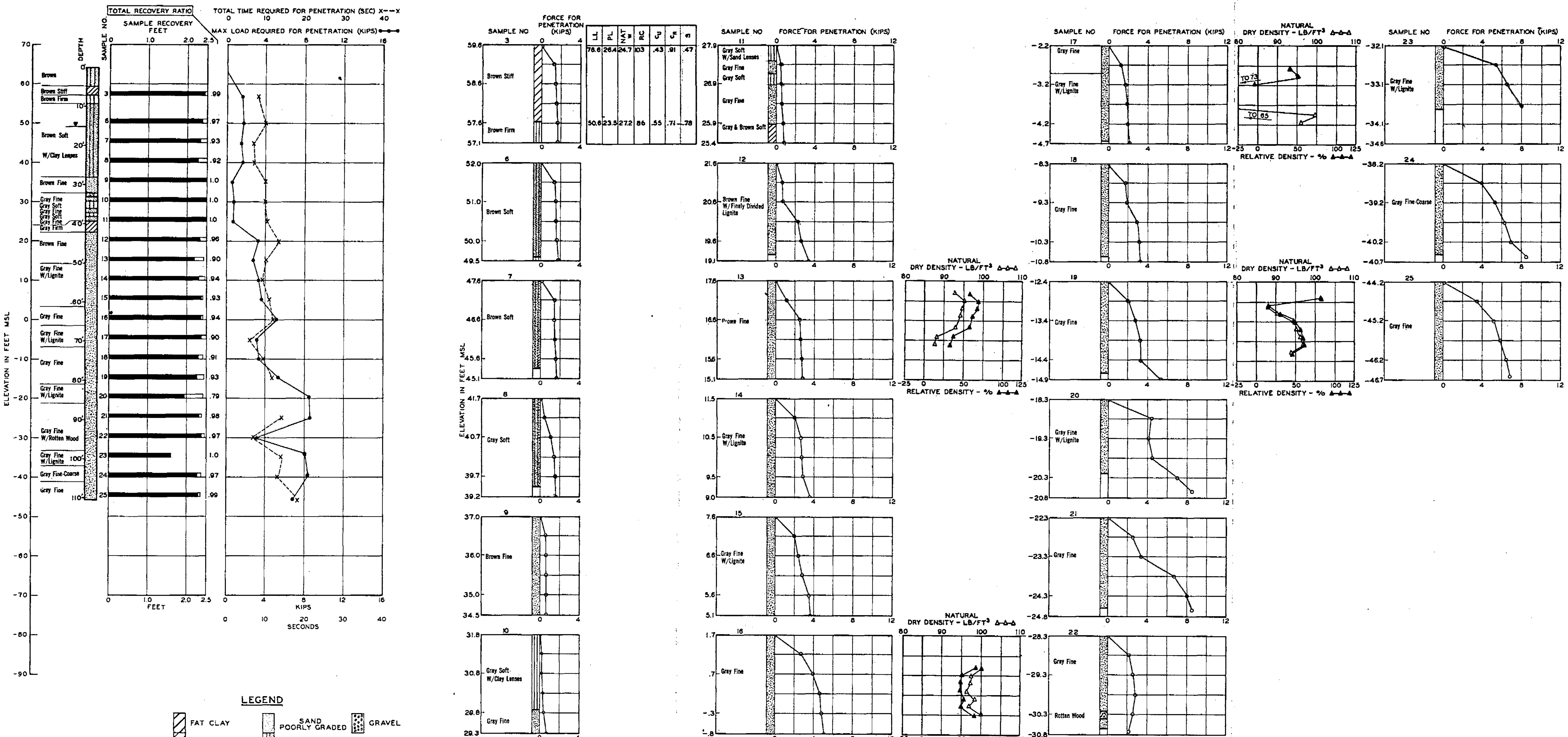


LEGEND

- FAT CLAY
- LEAN CLAY
- SILT SANDY SILT
- SAND POORLY GRADED
- SILTY SAND
- WATER TABLE
- GRAVEL

LL	LIQUID LIMIT
PL	PLASTIC LIMIT
NAT w	NATURAL WATER CONTENT
RC	RELATIVE CONSISTENCY – PERCENT
c_R	COHESION – REMOLDED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
c_U	COHESION – UNDISTURBED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
S	SENSITIVITY

LOG OF BORING R10-2A

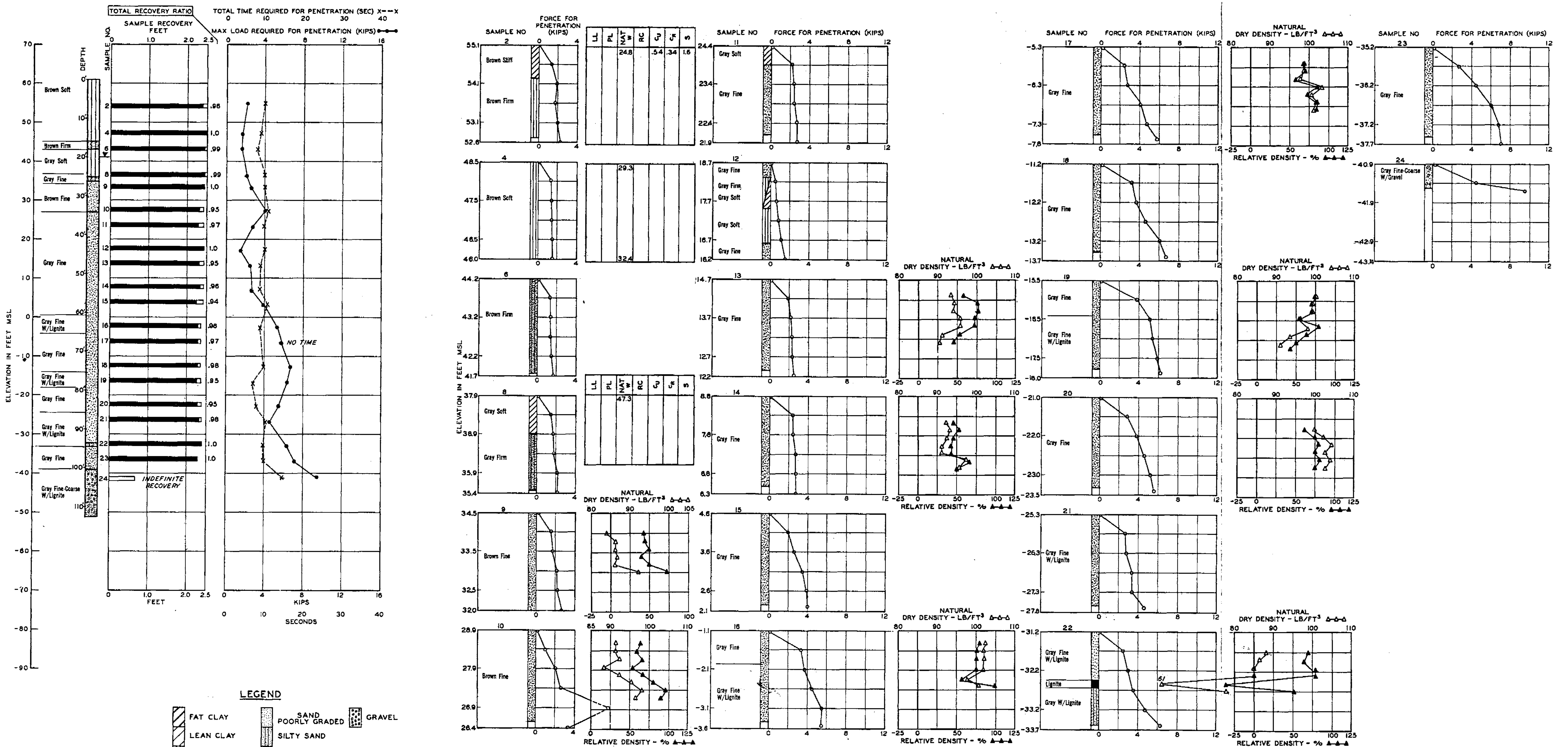


LEGEND

FAT CLAY
LEAN CLAY
SILT SANDY SILT
SAND POORLY GRADED
SILTY SAND
WATER TABLE
GRAVEL

LL	LIQUID LIMIT
PL	PLASTIC LIMIT
NAT w	NATURAL WATER CONTENT
RC	RELATIVE CONSISTENCY - PERCENT
c_R	COHESION - REMOLDED UNCONFINED COMPRESSION TEST (TONS PER SQ FT)
c_U	COHESION - UNDISTURBED UNCONFINED COMPRESSION TEST (TONS PER SQ FT)
S	SENSITIVITY

LOG OF BORING R 20-1

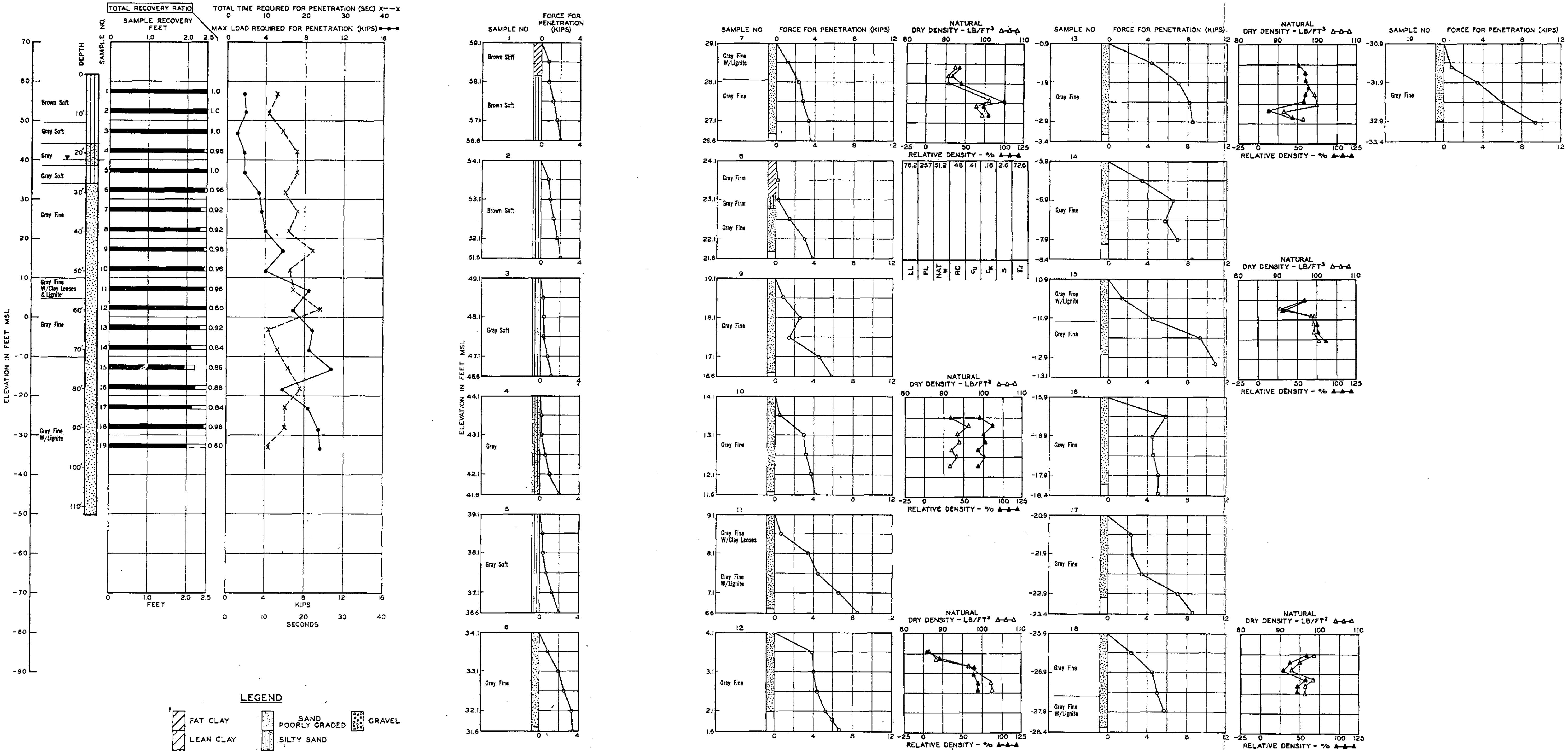


LEGEND

FAT CLAY
LEAN CLAY
SILT SANDY SILT
SAND POORLY GRADED
SILTY SAND
WATER TABLE
GRAVEL

LL	LIQUID LIMIT
PL	PLASTIC LIMIT
NAT w	NATURAL WATER CONTENT
RC	RELATIVE CONSISTENCY-PERCENT
c_R	COHESION - REMOLDED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
c_U	COHESION - UNDISTURBED UNCONFINED COMPRESSION TEST } TONS PER SQ FT
S	SENSITIVITY

LOG OF BORING R 30-1



LEGEND

FAT CLAY
LEAN CLAY
SILT
SANDY SILT
SAND POORLY GRADED
SILTY SAND
WATER TABLE
GRAVEL

LL	LIQUID LIMIT
PL	PLASTIC LIMIT
NAT w	NATURAL WATER CONTENT
RC	RELATIVE CONSISTENCY - PERCENT
c_R	COHESION - REMOLDED UNCONFINED COMPRESSION TEST (TONS PER SQ FT)
c_U	COHESION - UNDISTURBED UNCONFINED COMPRESSION TEST (TONS PER SQ FT)
S	SENSITIVITY

LOG OF BORING R34-1